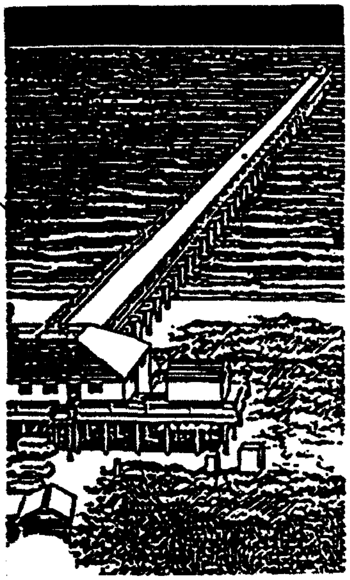
(2)

US Army Corps
of Engineers

TECHNICAL REPORT CERC-92-16

SCOUR PROBLEMS AND METHODS FOR PREDICTION OF MAXIMUM SCOUR AT VERTICAL SEAWALLS



by

Jimmy E. Fowler

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

December 1992

Final Report

Approved For Public Release; Distribution Is Unlimited

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20305-1000

Under Work Unit 31715

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY (*Leave blank*)

2. REPORT DATE
December 1992

3. REPORT TYPE AND DATES COVERED
Final report

4. TITLE AND SUBTITLE
Scour Problems and Methods for Prediction of Maximum Scour at Vertical Seawalls

5. FUNDING NUMBERS

6. AUTHOR(S)
Jimmy E. Fowler

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
USAE Waterways Experiment Station
Coastal Engineering Research Center
3909 Halls Ferry Road, Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER
Technical Report CERC-92-16

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
US Army Corps of Engineers
Washington, DC 20314-1000

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT (*Maximum 200 words*)
Laboratory experiments consisting of 22 tests were conducted in the 6-ft-wide wave flume at the US Army Engineer Coastal Engineering Research Center (CERC) to evaluate methods for estimating wave-induced scour depth (S) at vertical seawalls. Existing scour prediction methods range from rule-of-thumb estimates to semi-empirically derived equations. In the study, both regular and irregular waves were used to move sand with a mean diameter of 0.18 mm placed on the seaward side of a simulated vertical seawall. In the initial part of the study, 18 cases were run using irregular waves with various water depths, seawall locations relative to still-water level (swl), wave heights, and wave periods. All of the bottom profiles generated by the 18 irregular wave tests in the study supported a rule-of-thumb method, which states that maximum scour depth will be less than or equal to the incident unbroken deepwater wave height H_o, or $S/H_o \leq 1$. When additional data from other studies (which used regular waves exclusively) were considered, the rule of
(Continued)

14. SUBJECT TERMS
See reverse.

15. NUMBER OF PAGES
45

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT
UNCLASSIFIED

18. SECURITY CLASSIFICATION OF THIS PAGE
UNCLASSIFIED

19. SECURITY CLASSIFICATION OF ABSTRACT

20. LIMITATION OF ABSTRACT

13. (Concluded).

thumb did not hold for all cases. To examine the effects of regular versus irregular waves in movable-bed laboratory studies, four additional test cases were run using regular waves having comparable water depths, wave heights, wave periods, and seawall locations relative to swl to four of the irregular wave test cases. In each of the four regular wave cases, scour depth exceeded scour depths associated with comparable irregular wave tests. On the average, scour depth increased by approximately 15 percent with regular water conditions. Although this constitutes only a minimal effort to examine the differences between profiles generated by regular and irregular waves, this may account for many of the observed laboratory exceptions to the $S/H_0 \leq 1$ rule of thumb.

The irregular wave test results were also used to develop a dimensionless equation for estimation of wave-induced scour depth in front of vertical seawalls:

$$\frac{S_{max}}{H_o} = \sqrt{22.72 d_w / L_o + .25}$$

For the above equation, d_w is the pre-scour depth of water at the base of the wall and L_0 is the deepwater wave length. *Use of the above equation is limited to cases where* $-0.011 \leq d_w / L_0 \leq 0.045$ *and* $0.015 \leq H_0 / L_0 \leq 0.040$. The last condition restricts the equation to use with waves which are typical of most storms. Based on laboratory results obtained from the present study, it is recommended that where possible, the conservative $S/H_0 \leq 1$ rule of thumb should be used in the design of vertical seawalls. For cases where more precise estimation of potential scour depth is required, the equation presented above should be used subject to the noted constraints.

14. (Concluded).

Coastal
Flume studies
Irregular waves
Moveable bed model
Physical model
Scour prediction
Scour
Seawall
Sedimentation
Vertical seawall

PREFACE

This report was prepared by the US Army Engineer Waterways Experiment Station (WES), Coastal Engineering Research Center (CERC), and is the result of work performed under Coastal Research and Development Program Work Unit 31/15, "Laboratory Studies on Scour." This research was authorized and funded by Headquarters, US Army Corps of Engineers (HQUSACE), and was conducted by Dr. Jimmy E. Fowler, Wave Processes Branch (WPB), Wave Dynamics Division (WDD), CERC, under the general supervision of Dr. James R. Houston, Director of CERC; Mr. Charles C. Calhoun, Jr., Assiatant Director, CERC, Mr. C. E. Chatham, Chief, WDD, and Mr. D. G. Markle, Chief, WPB. The HQUSACE Technical Monitors for this research were Messrs. J. H. Lockhart, J. G. Housley, and B. W. Holiday.

The report was prepared by Dr. Fowler. The author acknowledges the contributions of the following: Dr. Steven A. Hughes, Research Hydraulic Engineer, CERC; Messrs. L. A. Barnes and J. E. Evans, Engineering Technicians, CERC; and Ms. J. A. Denson and Mr. R. R. Sweeney, Contract Students, CERC.

Director of WES during preparation and publication of this report was Dr. Robert W. Whalin. Commander of WES was COL Leonard G. Hassell, EN.

CONTENTS

LIST OF TABLES

LIST OF FIGURES

CONVERSION FACTORS, US CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

US customary units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9*	Celsius degrees
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	2.54	centimetres
pounds (force)	4.4482205	newtons
pounds (mass)	0.4535929	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

SCOUR PROBLEMS AND METHODS FOR PREDICTION OF MAXIMUM SCOUR AT VERTICAL SEAWALLS

PART I: INTRODUCTION

General

1. One of the most common coastal protection structures is the seawall, the majority of which are vertical faced. Under certain wave and/or current conditions the base, which supports the seawall, can be eroded and partial or total failure of the protective structure can occur. It is very costly to repair these structures; therefore, proper initial design and construction methods are imperative. To properly design seawalls, it is important to be able to estimate the potential amount of scour or loss of sediment at the toe. In most coastal environments, waves, tides, and currents interact resulting in a hydraulically complex situation. A physical model is often required to study and evaluate the stability and functional characteristics of the various designs and operating methods for seawalls.

Purpose

2. The purpose of this report is to review existing methods for scour prediction at vertical seawalls, to present results from a laboratory study formulated to study scour at vertical seawalls, to develop improved scour prediction techniques, and to delineate which scour prediction methods are most appropriate for various field applications.

Background

3. Scour at the sea-side toe of a vertical seawall has been the subject of research efforts for many years. To adequately study this problem, researchers must address the various effects of waves, wind, tide, currents, and storm surge on both the structure itself and the bed on which the structure resides. Prediction methods for scour at vertical walls vary from using rules of thumb to semi-empirically derived equations. When complex prototype situations are to be modeled (such as might exist where interactions between water levels, currents, and waves are involved), existing numerical prediction methods may be deemed inadequate, and physical model studies may be used. When properly designed and operated, these models can be used to accurately reproduce hydraulic conditions and to study/evaluate stability and functional characteristics of various proposed designs.

4. For additional discussion on the problem of scour at vertical seawalls or other vertical wall structures, consult Kraus (1988), Athow and Pankow (1986), Powell (1987), and Herbich et al. (1984). The problem associated with a vertical structure in the presence of an oscillatory wave climate is amplified because of reflected wave energy which is inherent to such a structure. The net result of wave reflection usually is to increase the depth to which the wave can influence the bottom. In most cases where scour at vertical seawalls has caused failure, local foundation materials are eroded beyond or near the bottom of the structure (Figure 1). Following this, impinging waves exert pressure on the upper part of the structure and failure occurs when the sediment at the toe of the wall is scoured to the point where its resisting ability is overcome by wave forces, gravity, and back pressures exerted by fills on the shore side of the structure.

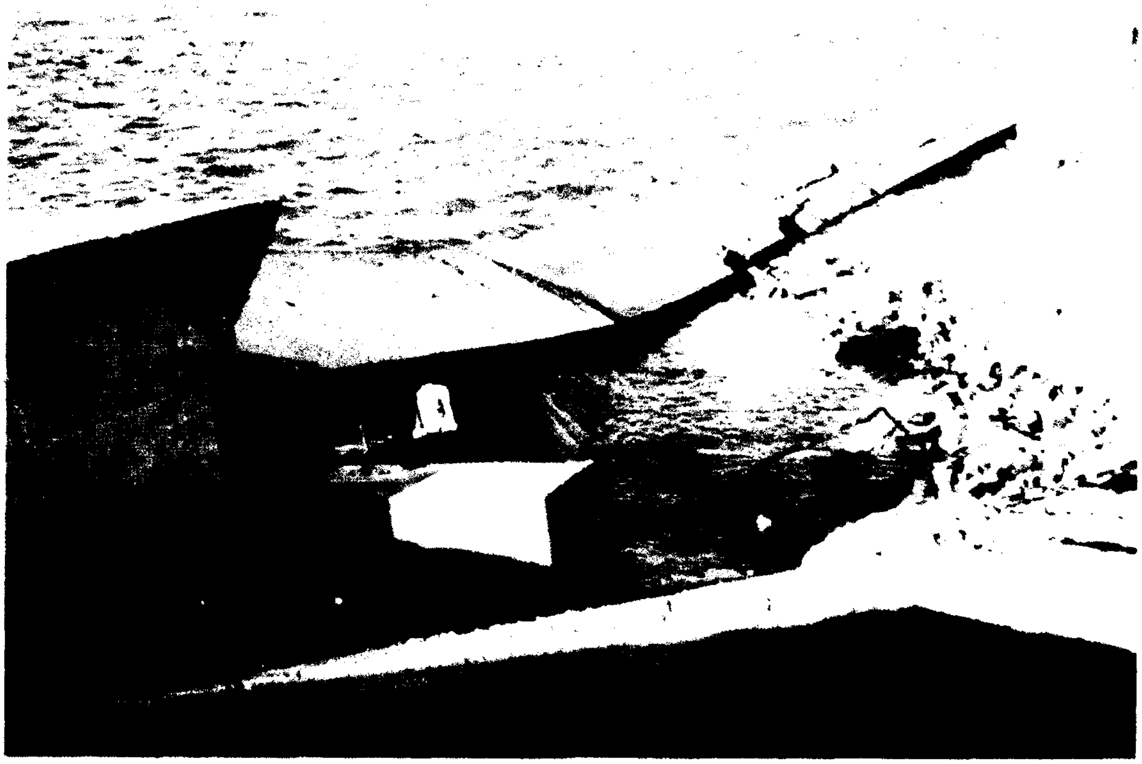


Figure 1. Scour problems at vertical seawalls

5. Another case where scour at vertical walls is a problem occurs as a result of tidal- or river-related currents. In this case, there may be some wave action (typically from boat or ship traffic) but the predominant scouring force is the current at the base of the structure. In the scouring mode, sediment is moved from the base by the current and for one reason or another

is not replaced. When this occurs over an extended period of time, the structure's foundation support is removed and the structure collapses from its own weight or the load exerted by its landside material. To combat this, stone blankets often are used along the sea-side toe to minimize scour. Pure flow-induced scour is not addressed in this study.

Organization of Report

6. A brief description of coastal scour problems at vertical seawalls is presented in Part I. Part II is a survey of various prediction methods and studies associated with scour at vertical seawalls. Part III contains a description of laboratory facilities and test and analysis procedures associated with the study reported herein. Part IV presents study results. Part V discusses results presented in Part IV and contains a summary which includes recommendations for scour prediction methods and additional research requirements. Appendix A is a listing of nomenclature used in the report.

PART II: LITERATURE SURVEY

Scour Prediction Methods for Vertical Seawalls

7. For most scour problems, the primary concern is the amount and location of scour which will occur, both in terms of area, depth, and proximity to the seawall toe. Depth of scour S has been studied by numerous investigators and a general relationship may be given as a function (F_1)

$$S = F_1(\rho, \rho_s, D, \omega, d, U_o, \nu, T, L, X, H) \tag{1}$$

For the above,

- ρ = fluid density
- ρ_s = sediment density
- D = mean sediment diameter
- ω = sediment fall speed
- d = depth
- U_o = bed or boundary velocity
- ν = fluid kinematic viscosity
- T = wave period
- L = characteristic length of structure
- X = position of seawall relative to shoreline
- H = wave height

Where scour has been determined to be an onshore-offshore mechanism, with little or no longshore movement, i.e., two-dimensional (2-D), the contribution from some of the above parameters is minimal and these may be omitted. Researchers have typically developed non-dimensional relationships for predicting scour, expressing relative scour in terms of incident wave height as S/H. The following chapter briefly describes various prediction methods, laboratory studies, and field studies concerning prediction of wave-induced scour at vertical structures. For additional discussion on prediction of scour at vertical seawalls, consult Herbich et al. (1984), the Shore Protection Manual (1984), Jones (1975), Walton and Sensabaugh (1979), Barnett (1987), Powell (1987), and Kraus (1988).

Rule-of-Thumb Methods

8. Based primarily on 2-D laboratory testing and a limited number of field observations, a rule of thumb states that maximum scour depth below the natural bed S_{max} is roughly less than or equal to the height of the unbroken deepwater wave height H_o (i.e., $S_{max}/H_o \leq 1$).

9. Dean (1986) used the "principle of sediment conservation" to develop an "approximate principle" to predict the volume of local scour that would occur during a 2-D situation (e.g., storm-dominated, onshore-offshore sediment transport). Dean proposed that the total volume of sediment lost from the front of a structure would be equal to or less than the volume that would have been lost if the structure had not been constructed. In other words, the amount (volume) of scour immediately in front of the structure would be less than or equal to the volume of sediment that would have been provided from behind the wall, had it not been there. Dean does not provide a method for estimating no-structure scour, and would rely on field measurements or engineering judgements based on local observations.

Semi-Empirical Methods

10. Jones (1975) used a number of limiting assumptions (including an infinitely long structure and perfect reflection from the wall) to derive an equation for estimation of scour depth. Jones' equation relates ultimate scour depth S to breaking wave height H_b and X_s, the dimensionless location of the seawall relative to the intersection of mean sea level (msl) and the beach profile. Jones defined X_s as follows:

$$X_s = \frac{X}{X_b} \qquad (2)$$

where X is the distance of the seawall from the point of wave breaking and X_b is the distance of the point of wave breaking from the intersection of msl with the pre-seawall beach profile (see Figure 2). Both distances are derived for the pre-seawall condition and may be determined by the commonly used method presented in the Shore Protection Manual (1984).

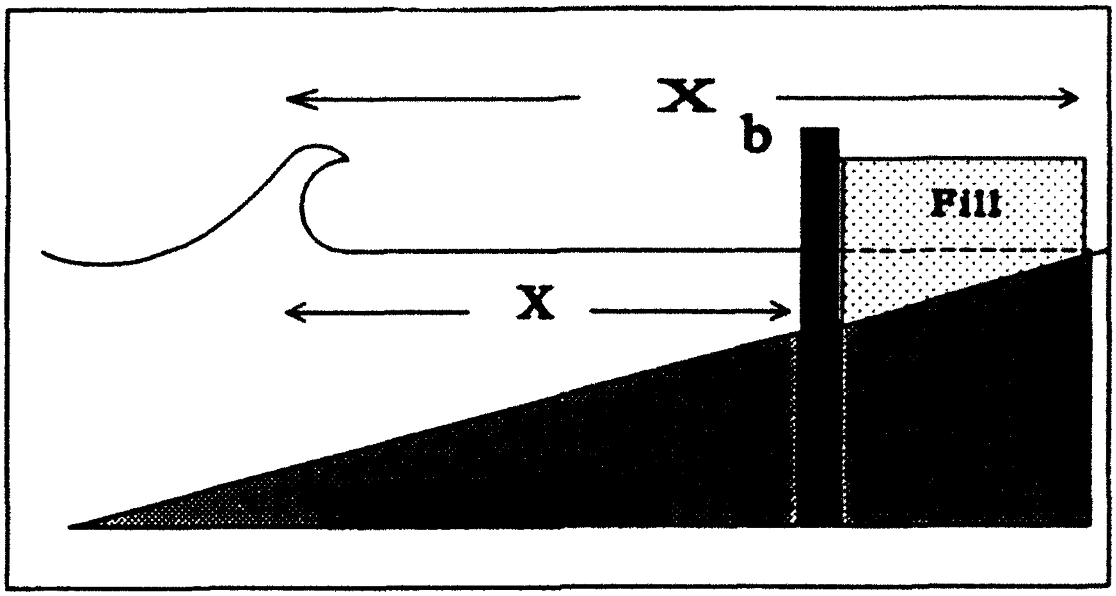


Figure 2. Definition sketch for Jones' method

When the location of the toe of the seawall coincides with the location of msl, $X_s = 1$. The following empirical equation was proposed for prediction of maximum scour depth:

$$\frac{S_{max}}{H_b} = 1.60\ (1 - X_s)^{2/5} \tag{3}$$

11. Using small-scale 2-D laboratory studies, Song and Schiller (1973)produced a regression model that predicts relative ultimate scour depth expressed as S_{max}/H_o . The relative ultimate scour was given as a function of relative seawall distance and deepwater standing wave steepness:

$$\frac{S_{max}}{H_o} = 1.94 + 0.57\ \ln(X_s) + 0.72\ \ln(H_s/L_s) \tag{4}$$

For the above, ln is the natural logarithm. Figure 3 displays this relationship for various values of relative ultimate scour depth.

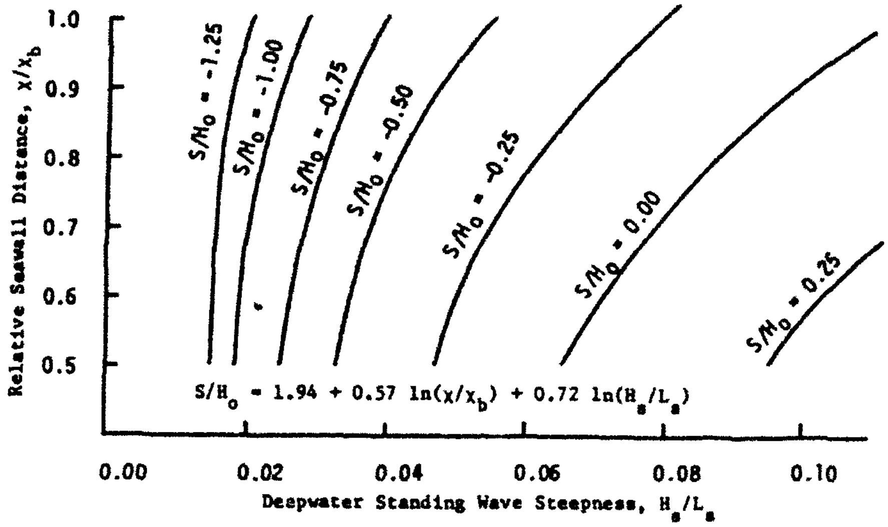


Figure 3. Plot relating relative scour depth to wave steepness and relative seawall distance (after Hales (1980))

12. The following equation was developed by Herbich and Ko (1968) using limited 2-D laboratory data to predict the ultimate depth of scour S_{max} for conditions where waves do not break prior to impacting the structure:

$$S_{max} = (d-a/2)\left((1-C_r)\ u_*\left[3/4\ C_D\ \rho\ \frac{\cot\phi}{D\ (\gamma_s - \gamma)}\right]^{1/2} - 1\right) \tag{5}$$

In the above,

$$a = H_i + H_r \tag{6}$$

$$C_r = \frac{H_r}{H_i} \tag{7}$$

and

C_D - drag coefficient of the particle
ϕ - bed material angle of repose
H_i - incident wave height
H_r - reflected wave height
u_* - local velocity parallel to the bottom
γ - fluid specific weight

γ_s = sediment specific weight

D = mean sediment diameter

d = depth

The above method requires knowledge of a relationship between incident and reflected wave heights, either through measurements made in the laboratory or, when available, through published values of C_r .

Laboratory Studies to Investigate Scour at Seawalls

13. Sato, Tanaka, and Irie (1968) studied scour in front of seawalls for both normal and storm beach profiles. In their study, seawall inclination (angle face of seawall makes with horizontal), grain size, beach slope, and wave conditions were varied using monochromatic waves in a 2-D facility. Five different types (modes) of scour were identified as described below:

Type 1 - Rapid initial scour followed by a gradual accretion of material

Type 2 - Rapid initial scour leading to beach stability

Type 3 - Rapid initial scour giving way to slower, but more prolonged erosion

Type 4 - Continuous gentle scour

Type 5 - Continuous gentle accretion

In addition to identifying the different scour modes, Sato, Tanaka, and Irie reached the following conclusions:

a. Relative scour depth S/H_o can be larger than unity for flatter (non-storm) waves but for storm waves with steepness between 0.02 to 0.04, the relative scour depth was equal to unity.

b. Relative scour depth decreased with decreasing relative median grain size d_{50}/H_o .

c. Maximum scour depth for storm waves occurred when the wall was located at either the shoreline or just landward of the plunge point.

d. Maximum scour depths occurred for the Type 3 classification of scour, which is characterized by rapid initial scouring giving way to slower, more prolonged erosion associated with storm wave conditions.

e. Maximum scour depths occurred for seawall inclinations of 90 deg* and initial beach slope had little effect for the range of conditions tested.

14. Chesnutt and Schiller (1971) conducted approximately 50 tests in two different wave flumes to investigate scour in front of seawalls along the Texas Gulf Coast. The sand used in their study was Texas beach sand having mean diameter of 0.17 mm . The study investigated scour depths associated with various wave conditions, beach slope, seawall locations, and seawall inclination. The more significant findings of this study included:

---

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 4.

a. Maximum scour is approximately equal to the deepwater wave height for the range of conditions tested. Wave steepnesses ranging from 0.003 to 0.036 were run for the cases where the seawall was at a 90 deg (vertical) inclination.
b. Maximum scour for seawall location occurs in the range of $0.5 < X_s < 0.67$, with X_s as previously defined.
c. Maximum scour depth increases with increase in wave height.
d. Maximum scour depth decreases with decrease in angle of inclination of the seawall, or as the angle the face of the seawall makes with the horizontal decreases.
e. Maximum scour depth decreases with decrease in beach slope.

15. Barnett (1987) used an empirical eigenfunction analysis on 2-D laboratory data using regular waves on a fine sand and some limited prototype data to examine the effects of seawalls on beach profile response. The eigenfunction analysis method has been used successfully by others such as Kriebel, Dally, and Dean (1986). For simplicity, the analysis method is not discussed here. In Barnett's tests, erosive wave conditions without a seawall were compared with wave conditions in similar tests with a seawall located at different positions relative to the intersection of the still-water level and the initial beach profile. Barnett's tests compared eroded volumes with and without the seawall to test Dean's approximate principle, which states that the eroded volume in front of the seawall will be less than or equal to the volume which would have been lost if the seawall had never been constructed. Basically, Barnett promoted the eigenfunction analysis as an efficient means of examining 2-D spatial and temporal profile variations and concluded that Dean's approximate principle was supported by results of the study. Barnett's work is included here primarily for comparison with results of this study.

16. In a study conducted at the US Army Engineer Waterways Experiment Station (WES) Coastal Engineering Research Center (CERC) during 1988-1989, a scaled physical model was used to validate selected movable-bed modeling guidance by simulating prototype scale wave-induced scour of sand in front of a concrete dike constructed at a 1:4 slope. The validated scaling guidance is appropriate for 2-D energetic (wave action) erosion models and is presented in Part III of this report. Near prototype data used were obtained from the large wave tank tests done by Dette and Uliczka (1987) at the University of Hannover in Germany during 1985-1986. In conjunction with validation tests, the scaling guidance was used in two additional cases to simulate scour in front of a vertical wall placed on top of the concrete dike (Figure 4). Tests were designed to duplicate initial beach profiles and wave conditions used in validation tests without the vertical wall. Based on results obtained using both regular and irregular wave trains, Dean's approximate principal was supported by the two cases tested.

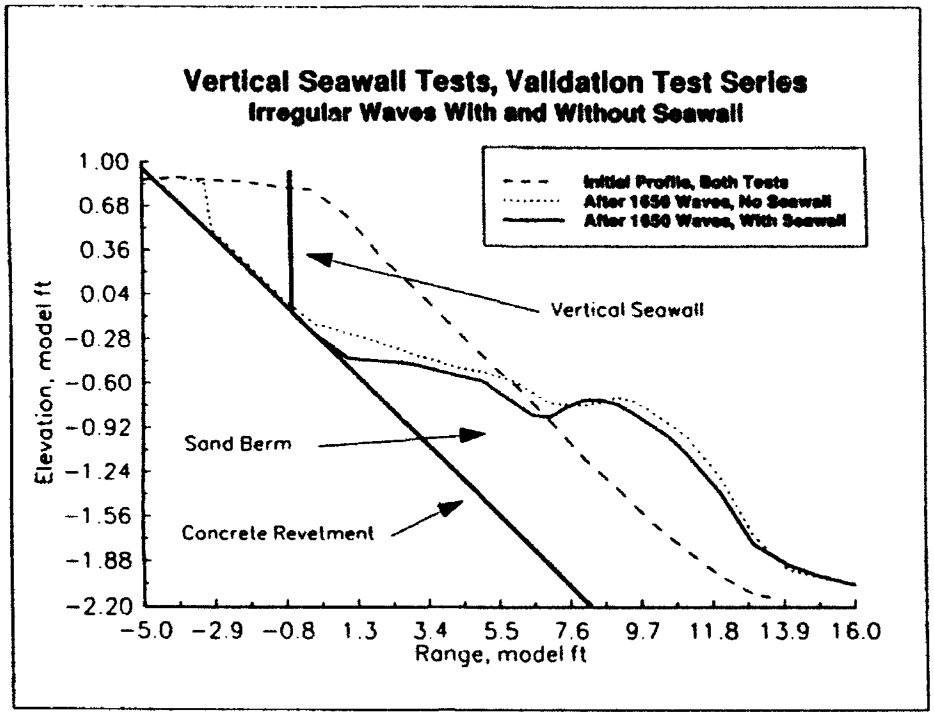


Figure 4. Vertical wall tests done in conjunction with validation tests

Field Studies

17. Sato, Tanaka, and Irie (1968) also presented field data obtained following a storm that significantly scoured foundations fronting seawalls at the Port of Kashima. Their data supported the findings listed in paragraph 13, particularly the finding that maximum scour depth S_{max} is less than or equal to deepwater significant wave height. Measured scour depths at seawalls showed that maximum scour depth under storm conditions was nearly equal to the maximum significant deepwater wave height H_s observed during the storm.

18. Sawaragi and Kawasaki (1960) compiled field data on erosion in front of seawalls at eight sites in the Sea of Japan. The data obtained covered a period during which the seawalls were impacted by three significant storms. Analysis of the data led the authors to conclude that the maximum depth of scour is approximately equal to the wave height in deep water and that the location of maximum scour is related (proportional) to location of the point of breaking of incident waves.

19. Sexton and Moslow (1981) obtained data along seawall-backed beaches at Seabrook Island, South Carolina to examine scour and subsequent recovery following the September 1979 attack of Hurricane David. The beach in front of one concrete seawall experienced a scour depth of 0.64 m and overtopping also

caused some scour on the landward side of the seawall. Since maximum deepwater wave heights exceeded this value considerably, the $S/H_o \leq 1.0$ rule of thumb is apparently supported here as well.

20. Walton and Sensabaugh (1979) examined field data associated with scour that was observed in Panama City, Florida following Hurricane Eloise in September 1975. From their observations, it was noted "that apparent seawall scour observed at Panama City ... was considerably less than the maximum predicted by the rule of thumb." Additionally, the authors stated that "most seawalls with cap elevations less than 10 ft above grade experienced a maximum of 2-3 feet of scour." This observation was for unprotected beaches that fronted seawalls in the area studied.

Summary

21. One of the problems associated with determining maximum scour depth in the field is related to the difficulties associated with obtaining immediate or "unhealed" measurements following storm events. If field measurements are made a significant amount of time following the storm, there is some risk that accretion may occur and lower the S_{max}/H_o ratio. Because of this, the majority of techniques for prediction of maximum scour depth are empirical in nature and derive their merit from laboratory studies "validated" by limited field data or observations of scour following severe storms. Each of these cases generally is derived for conditions which support a predominantly onshore-offshore movement of sediment. Although this appears to present certain limitations for use of these methods, the available field data suggest that for maximum scour depth predictions, this should be a sufficient source.

PART III: FACILITIES, MATERIALS, AND PROCEDURES

Laboratory Facilities

22. The tests reported herein were done in CERC'S 6-ft-wide wave flume during the period May - November 1991. The flume is constructed of concrete and has glass viewing windows in the test section, which is located 245 ft from the wave board. The flume has the dimensions and capacities shown in Figure 5:

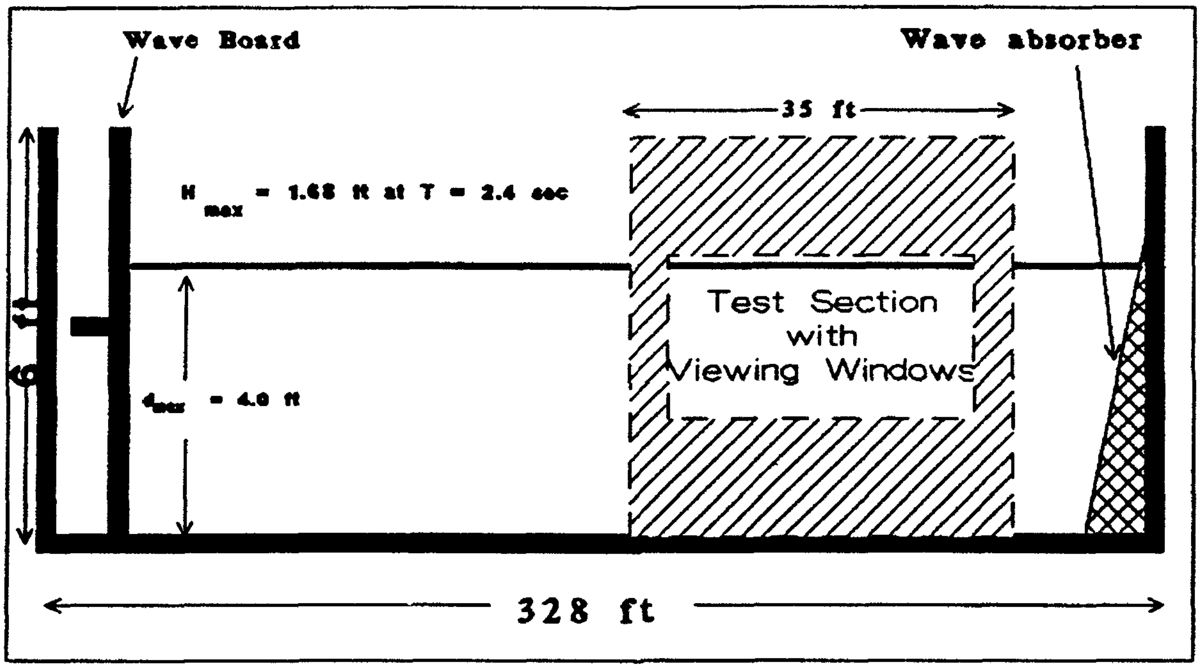


Figure 5. Characteristics of the 6-ft-wide flume facility

The wave machine used in the 6-ft flume is hydraulically operated and is constructed such that it may be used in either the flapper or piston mode and can generate waves of 0.5 m at maximum operating conditions. For the reported tests, the wave machine was operated in the piston mode to generate both regular (monochromatic) and irregular waves. Piston stroke and frequency for both regular and irregular waves are controlled using CERC software and a Micro-Vax I microcomputer. During operation of the wave machine, feedback from the piston motion and wave gages was actively monitored using a multi-channel oscilloscope as well as through digital recordings. Wave data were collected using both resistance and capacitance wave rods. An Automated Data

Acquisition and Control System (ADACS) designed and developed at WES (Turner and Durham 1980) was used to calibrate the wave rods and ensure correct measurements of wave heights. Six wave rods were used in two groups of three (Goda arrays) to allow calculation of reflected wave energy in both deep and shallow parts of the tank using the Goda method (Goda 1970). The wave rods were calibrated at the beginning of each test series to a tolerance of ±0.002 model feet. Figure 6 is a schematic of the ADACS used with the 6-ft-wide flume. To generate regular waves, a wave period and amplitude are specified and a sinusoidal data file with stroke and elapsed time is generated and used as the input signal to drive the wave machine. For irregular wave generation, CERC software is used to produce a piston stroke time series for the desired spectral parameters. Wave data were collected at a rate of 20 Hz and analyzed using both frequency and time domain techniques.

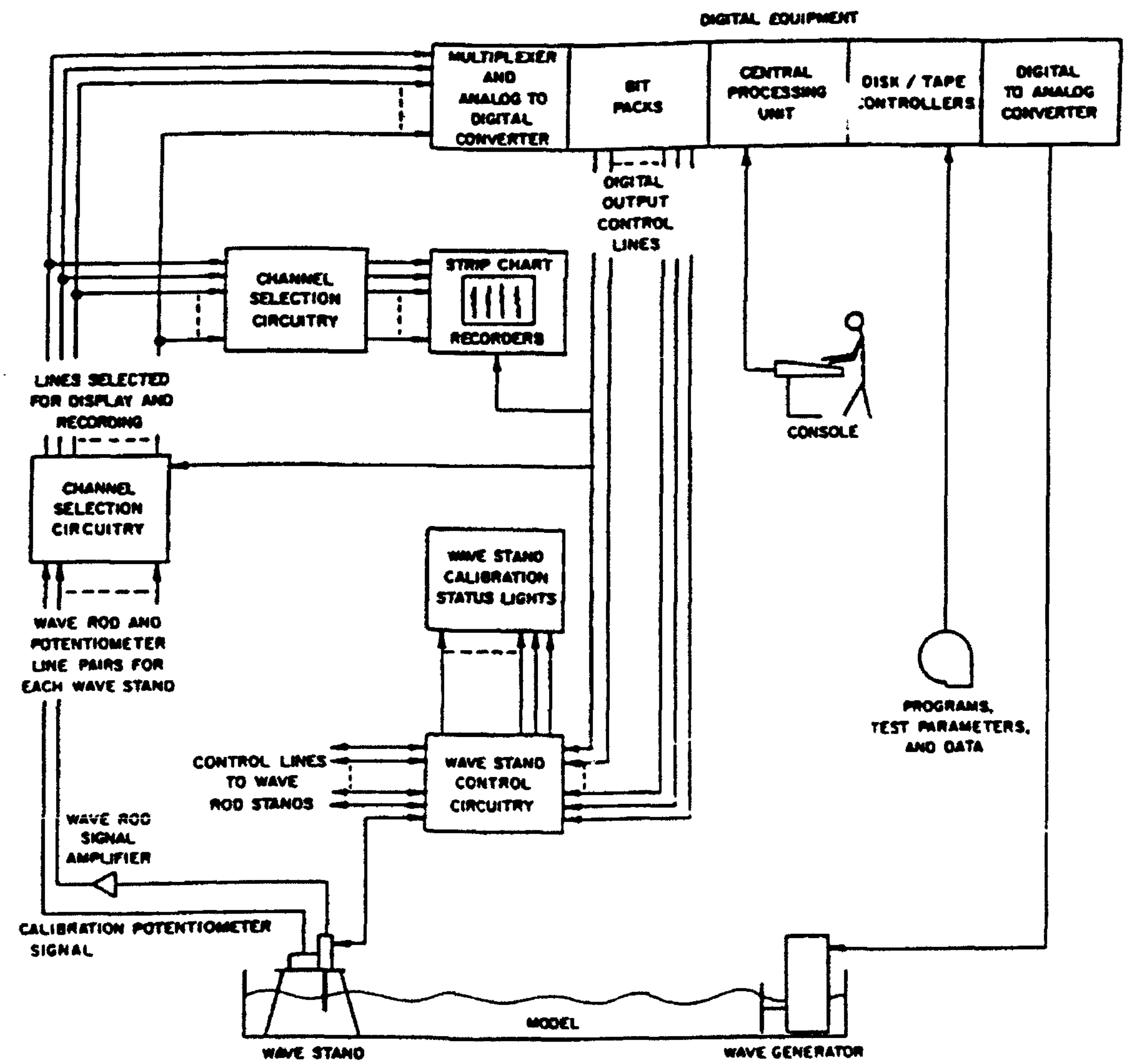


Figure 6. Schematic of ADACS for 6-ft-wide flume

Movable-Bed Model Scaling Criteria

23. In general, most researchers agree that two approaches/concepts are important for physically modeling how particles are moved from one location to another:

- a. Fall speed similarity.
- b. Incipient motion similarity.

Studies by Hughes and Fowler (1990) indicated that the guidance based on preserving fall speed similarity produces good results for energetic situations such as occur in the surf zone, where the turbulent energy associated with breaking waves dominates. Scaling by incipient motion criteria is more appropriate in situations where sediment transport is predominantly by bed load. The fall speed scaling guidance for simulation of sediment transport in very energetic environments, such as with wave-induced erosion, requires that the following criteria should be met:

Fall Speed Scaling Guidance for Wave-Energy-Dominated Erosion

1) Fall speed parameter ($H/\omega T$) similarity.

2) Time-scale-based Froude ($Fr = V/(gl)^{1/2}$) modeling.

3) Model is undistorted ($N_1 = N_x = N_y = N_z$).

4) Use fine sand (D = 0.08mm lower limit) as model sediment at largest possible scale ratio.

For the above:

- H = wave height
- ω = sediment fall speed
- T = wave period
- V = an appropriate velocity
- g = gravitational acceleration
- l = characteristic length
- N = Scale ratio

The subscripts 1, x, y, and z are characteristic length, length in x the direction, length in the y direction, and length in the z direction, respectively. Since the overwhelming majority of sediment transport for this study is by suspended load, the fall speed guidance was used to scale the model setup and test conditions.

24. The scaling guidance outlined above can be used to convert model values to corresponding prototype values. Using item number 1 above, similarity between model and prototype fall speed parameters is achieved when

$$\left[\frac{H}{\omega T}\right]_{model} = \left[\frac{H}{\omega T}\right]_{prototype} \tag{8}$$

For an undistorted model, $N_H = N_l$; therefore Equation 8 can be rewritten as

$$N_l = N_\omega \, N_T \tag{9}$$

For the above, N_ω, N_l, and N_t are the model-to-prototype ratios for sediment fall speed, length scale, and wave period, respectively. Froude scaling guidance for time is given by

$$N_t = \sqrt{N_l} \quad or \quad N_t^2 = N_l \tag{10}$$

where N_t is the model-to-prototype time scale ratio. Equations 8, 9, and 10 can be combined to yield a unique scaling guidance which satisfies the first two scaling criteria:

$$N_\omega = \sqrt{N_l} = N_t \tag{11}$$

The scaling relationship in Equation 11 can be used to convert model values to corresponding prototype conditions once a prototype sediment diameter (and corresponding fall velocity) is known. Figure 7 can be used to obtain fall speeds for various sand sizes. The Froude scaling criterion can be used to determine prototype wave period and elapsed time.

Model Sediment Characteristics

25. Fine quartz sand obtained from the Ottawa Sand Company in Ottawa, Illinois, having mean diameter of 0.13 mm with a specific gravity of 2.65 and a fall speed of 1.64 cm/sec, was used in all tests.

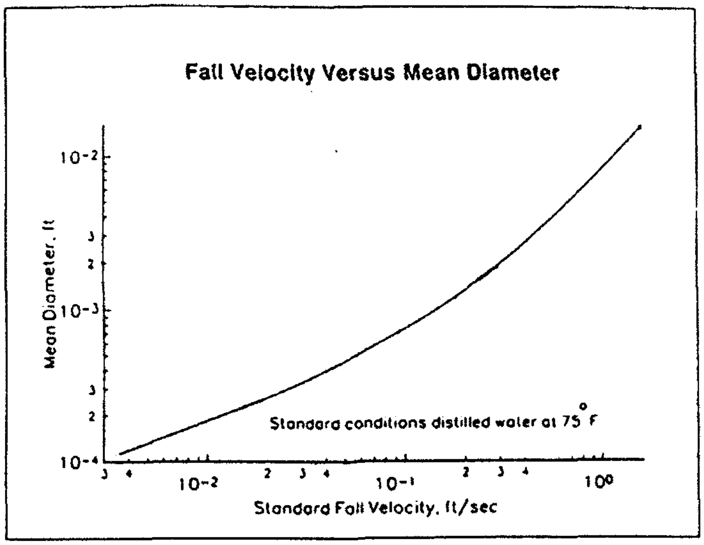


Figure 7. Fall velocity versus sand size (after Seabergh (1983))

Procedures

26. The procedure used for all tests was designed to simulate scour of sediment from a beach having mild initial slope (1V on 15H) in front of a seawall being impacted by storm waves approaching at a right angle to alignment of the vertical wall. The initial profile was smoothed to a 1V on 15H slope and then was surveyed. As previously stated, wave rods were calibrated prior to each test in order to ensure accuracy of wave data. Irregular waves then were generated in bursts of 300 sec with time for stilling allowed between runs to minimize reflection and re-reflection of wave energy. For all test conditions, waves broke well seaward or immediately in front of the seawall. The specific case of non-breaking waves was not addressed in this study, but is reported in Hughes and Fowler (1991).

27. Center-line profiles were surveyed at various points during the tests to allow determination of the "equilibrium" condition. The "equilibrium" condition was reached when successive profile surveys indicated little or no change. A graduated rod with a 2 in-diam circular foot pad was used to obtain all center line profiles as shown in Figure 8. Elevations were obtained along the profile at various (0.5- to 5-ft) intervals as required to reproduce the slope accurately. A benchmark elevation was taken at the beginning and end of every profile survey to ensure consistency between individual tests.

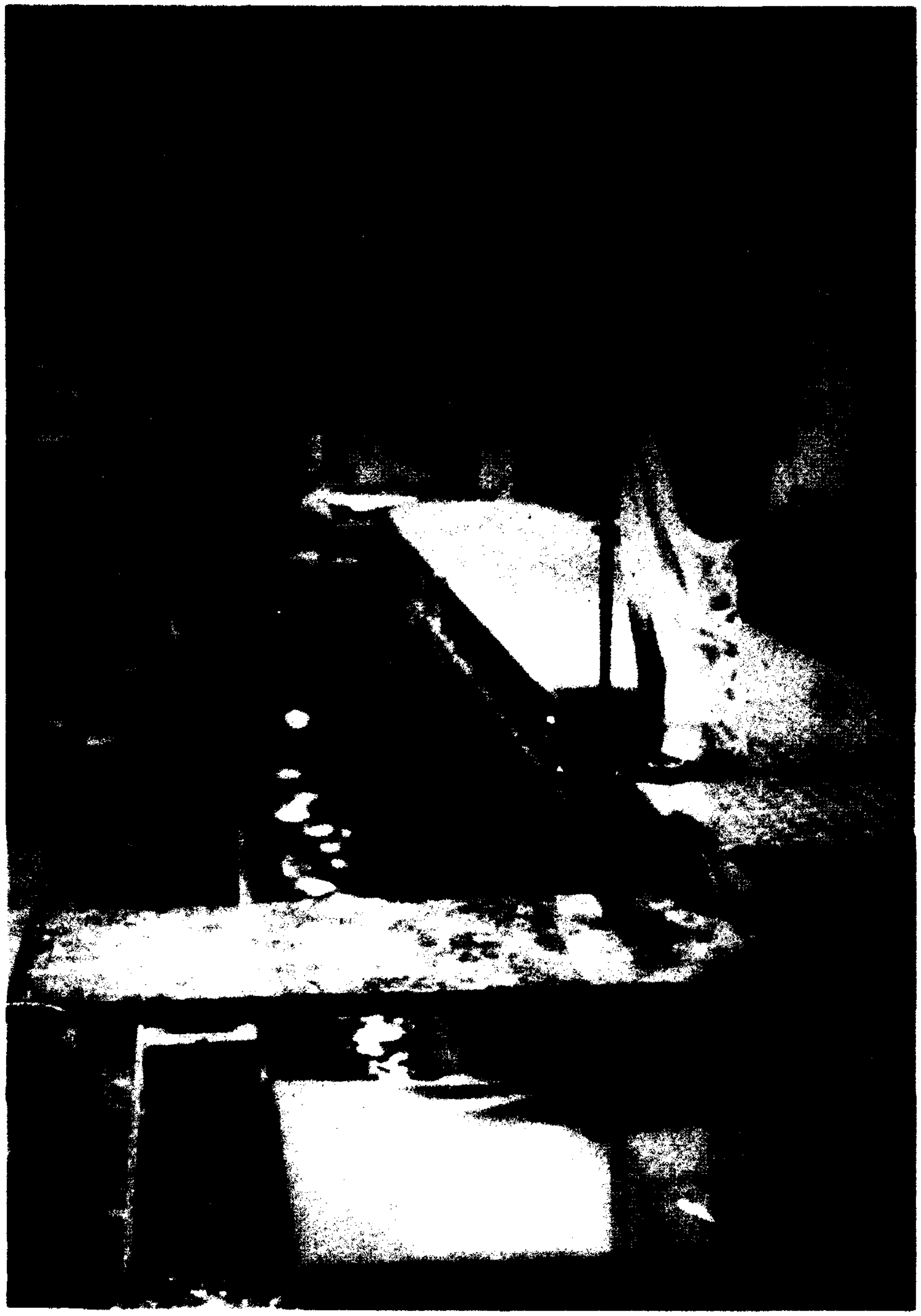


Figure 8. Photograph of procedure for taking profiles

PART IV: RESULTS

General

28. During the period 20 May - 25 November 1991, 22 tests were conducted using the facilities and setup as discussed in Part III. The initial 18 tests were conducted using irregular wave trains, while the final 4 tests were conducted using regular waves. In each of the regular wave cases, H_o corresponds to the average height of all waves generated, while the H_o for the irregular wave tests represents the significant wave height as measured in the deep section of the flume, approximately 20 ft from the wave board. Although wave heights measured in this portion of the flume may or may not be true representations of deepwater conditions, an analysis using linear wave theory indicates that the errors introduced are conservative (since the deepwater wave height would be slightly larger) and amount to less than 10 percent. Tables 1 and 2 summarize pertinent test parameters for the irregular waves and regular waves, respectively. Figure 9 provides definitions for information contained in Tables 1 and 2. Bottom profiles were obtained during all tests at various time intervals to document profile change and determine "equilibrium conditions." Figure 10 shows a typical sequence of bottom profiles surveyed during the tests. A complete data set is available upon request in an unpublished document. Scour depths listed under "Maximum Seawall Scour Depth" in Tables 1 and 2 are maximum values of scour measured immediately seaward of the seawall. In some tests, this value did not correspond to the maximum depth of erosion, which is given in the "Maximum Erosion" column, with locations of maximum eroded depth given relative to the seawall itself. The modeling guidance presented in Part III can be used to relate test results to reasonable prototype scale values. As an example, for a prototype having mean sand size of 0.35 mm, the guidance yields 1 to 7.5 for the geometric scale and 1 to 2.7 for the time scale.

Table 1

Summary of Irregular Wave Test Conditions*

Test #	Seawall Location wrt msl ft	Water Depth d, ft	Deepwater Wave Height H_o, ft	Wave Period T_o, sec	Wave Length L_o, ft	Fall Velocity ω, ft/sec	Maximum Scour Depth S_{max1}, ft	Max Erosion Location X_{max} ft seaward	Scour Depth at Seawall S_{max}, ft	$\frac{S_{max}}{H_o}$
S1	0.0	4.0	0.693	1.97	19.95	0.063	-0.44	0.0	-0.44	0.63
S2	0.0	3.8	0.660	1.97	19.95	0.063	-0.27	0.0	-0.27	0.41
S3	0.0	4.0	0.683	1.97	19.95	0.063	-0.50	0.0	-0.50	0.73
S4	0.0	4.0	0.783	2.49	31.87	0.063	-0.63	0.0	-0.63	0.80
S5	0.0	3.7	0.843	1.97	19.95	0.063	-0.21	5.0	-0.08	0.37
S6	0.0	3.7	0.887	2.45	30.85	0.063	-0.27	1.0	-0.26	0.30
S7	0.0	3.7	0.800	1.97	19.95	0.063	-0.27	1.0	-0.22	0.39
S8	0.0	4.0	0.641	1.97	19.95	0.063	-0.58	0.0	-0.58	0.90
S9	+3.0	3.8	0.983	2.43	30.35	0.063	-0.40	0.0	-0.40	0.41
S10	+3.0	3.8	0.683	1.93	19.15	0.063	-0.51	0.0	-0.51	0.75
S11	+3.0	3.7	0.699	1.97	19.95	0.063	-0.47	0.0	-0.47	0.67
S12	+3.0	4.0	0.681	1.99	20.35	0.063	-0.41	0.0	-0.41	0.60
S13	+3.0	4.0	0.896	2.40	29.61	0.063	-0.70	0.0	-0.70	0.78
S14	+3.0	3.7	0.950	2.45	30.85	0.063	-0.61	0.0	-0.61	0.64
S15	-3.0	3.8	0.882	2.45	30.85	0.063	-0.24	2.0	-0.11	0.27
S16	-3.0	3.8	0.655	1.97	19.95	0.063	-0.14	5.0	+0.03	0.21
S17	-3.0	3.7	0.876	2.48	31.61	0.063	-0.17	3.5	+0.09	0.19
S18	-3.0	4.0	0.660	1.95	19.54	0.063	-0.28	4.0	+0.15	0.42

Table 2

Summary of Regular Wave Test Conditions*

Test #	Seawall Location wrt msl ft	Water Depth d, ft	Deep water Wave Height H_o, ft	Wave Period T_o, sec	Wave Length L_o, ft	Fall Velocity ω, ft/sec	Maximum Scour Depth S_{max1}, ft	Max Erosion Location X_{max} ft seaward	Scour Depth at Seawall S_{max}, ft	$\frac{S_{max}}{H_o}$
M1	0.0	4.0	0.85	2.50	31.60	0.063	-0.90	0.0	-0.90	1.058
M2	0.0	4.0	0.65	1.98	19.95	0.063	-0.72	0.0	-0.72	1.090
M3	0.0	3.8	0.65	1.98	19.95	0.063	-0.55	0.0	-0.55	0.895
M4	0.0	3.8	0.85	2.50	31.60	0.063	-0.62	0.0	-0.62	0.735

* See Figure 9 for locations and definitions of measured values.

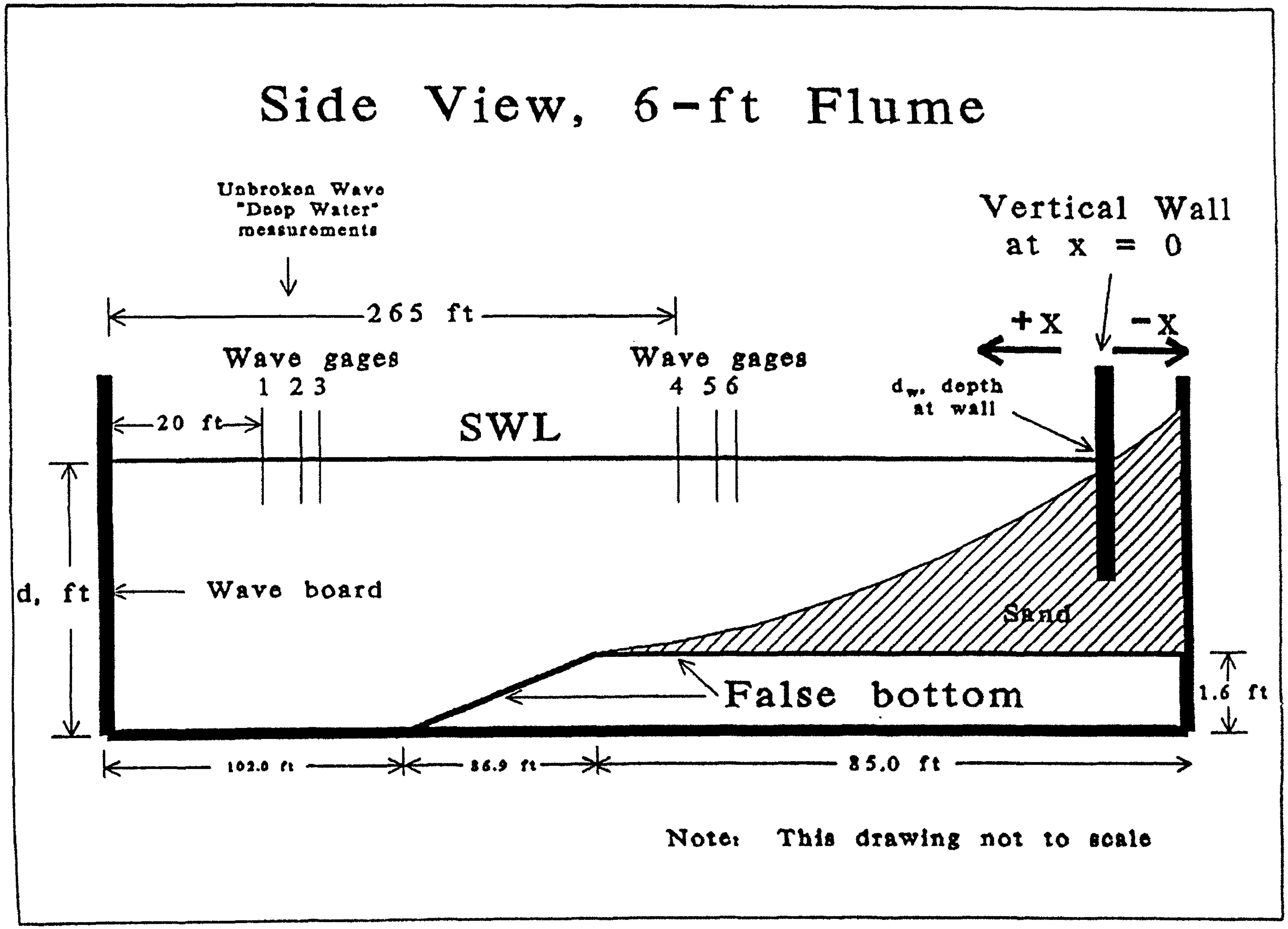


Figure 9. Schematic for interpretation of values in Tables 1 and 2

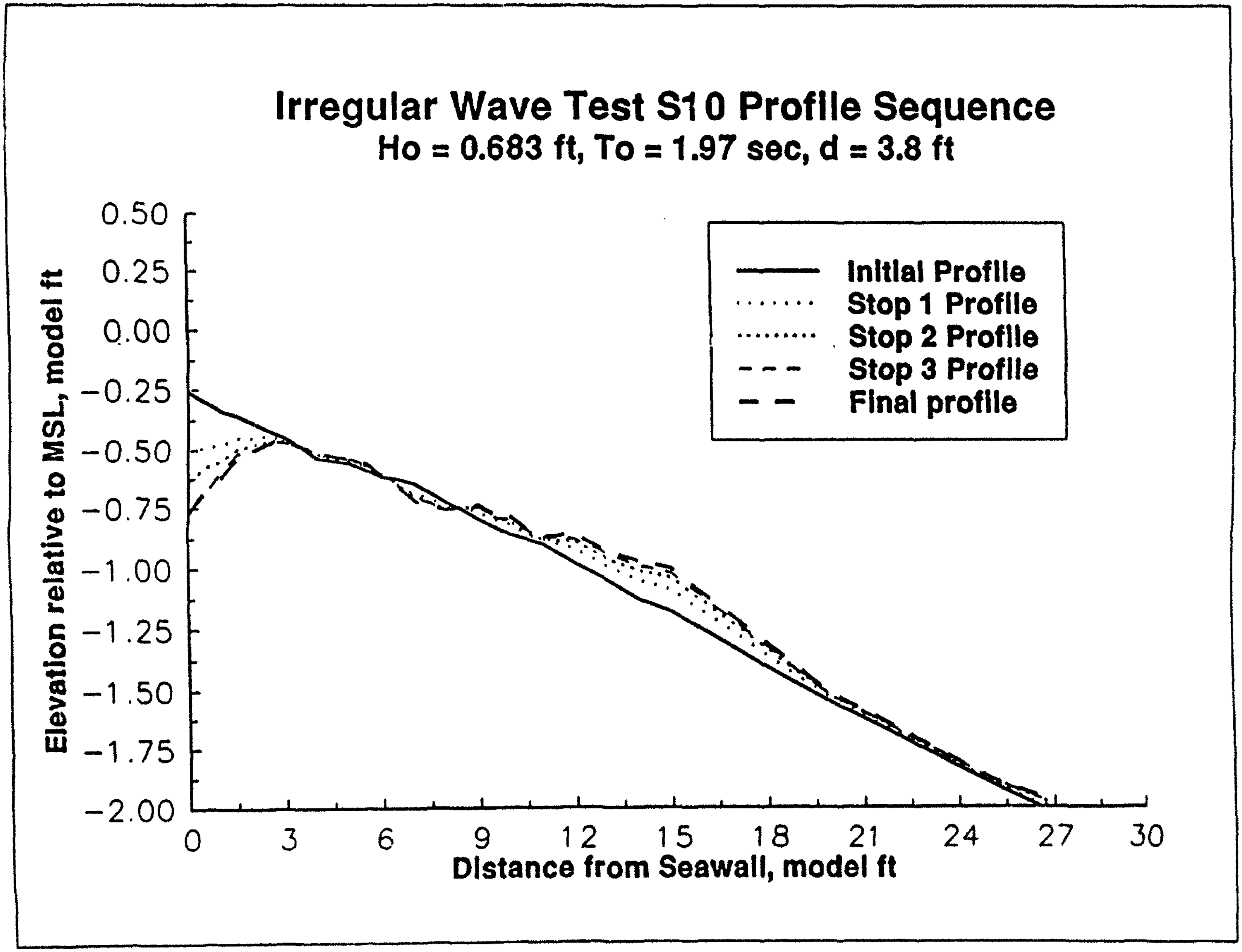


Figure 10. Typical bottom profile sequence

Maximum Scour Depth Versus Incident Wave Height

29. Figure 11 is a plot of maximum scour depths versus deepwater wave heights for all irregular wave tests. As can be seen, data from the laboratory tests conducted during this study support the rule of thumb which states that maximum scour will be less than or equal to the unbroken deepwater significant wave height. Data presented in Figure 11 also are coded to allow visualization of the effect of seawall location relative to the intersection of msl with the natural pre-scour profile. In the figure, solid circles correspond to tests conducted with the seawall located at the intersection of msl and the pre-seawall profile. Solid squares correspond to having the seawall located 3 model feet seaward of the msl location, and the solid diamonds show results when the seawall is located 3 model feet shoreward of the msl location.

30. Irregular and regular wave data from the tests are pooled with data from Barnett (1987) and Chesnutt and Schiller (1971) in Figure 12. As can be seen, some of the data from these studies exceed the $S/H_o \leq 1$ rule of thumb. Each of these cases was a laboratory study conducted using only regular waves on a sand bed.

31. To further examine the effect of regular versus irregular waves, an additional set of tests was run using regular waves with design parameters similar to some of the irregular tests listed in Table 2. Monochromatic waves seldom (if ever) are an accurate representation of wave conditions that exist in nature. This fact has probably contributed to significant design limitations (including over- and under-design of structures). Design guidance developed from models using uniform regular wave conditions typically represents the irregular waves that exist in Nature by a single statistical wave height parameter. This statistical parameter then is taken as being equivalent to the regular wave height in the design formulae.

Irregular Wave Parameters

32. Shallow-water waves in Nature are typically represented by statistical wave height parameters or energy-based parameters. These statistical parameters are representative of the wave climate during a period of time in which the wave process is assumed stationary. Typical statistical wave height parameters include: H_{avg} (mean wave height of all waves), H_{rms} (root mean square wave height), and $H_{1/3}$ (average of the highest one third of all waves). The primary energy-based wave height designator is H_{mo}, which is directly related to energy contained in the wave spectrum. H_{mo} is approximately equal to $H_{1/3}$ for deepwater waves but can be significantly different for shallow-water waves (Thompson and Vincent 1984, Hughes and Borgman 1987). Results from these physical model tests indicated that significant wave height is the best irregular wave design parameter for

matching results based on uniform regular wave tests. This finding agrees with work done previously by others (Mimura, Otsuka, and Watanabe 1986) during investigations to predict the threshold of movement.

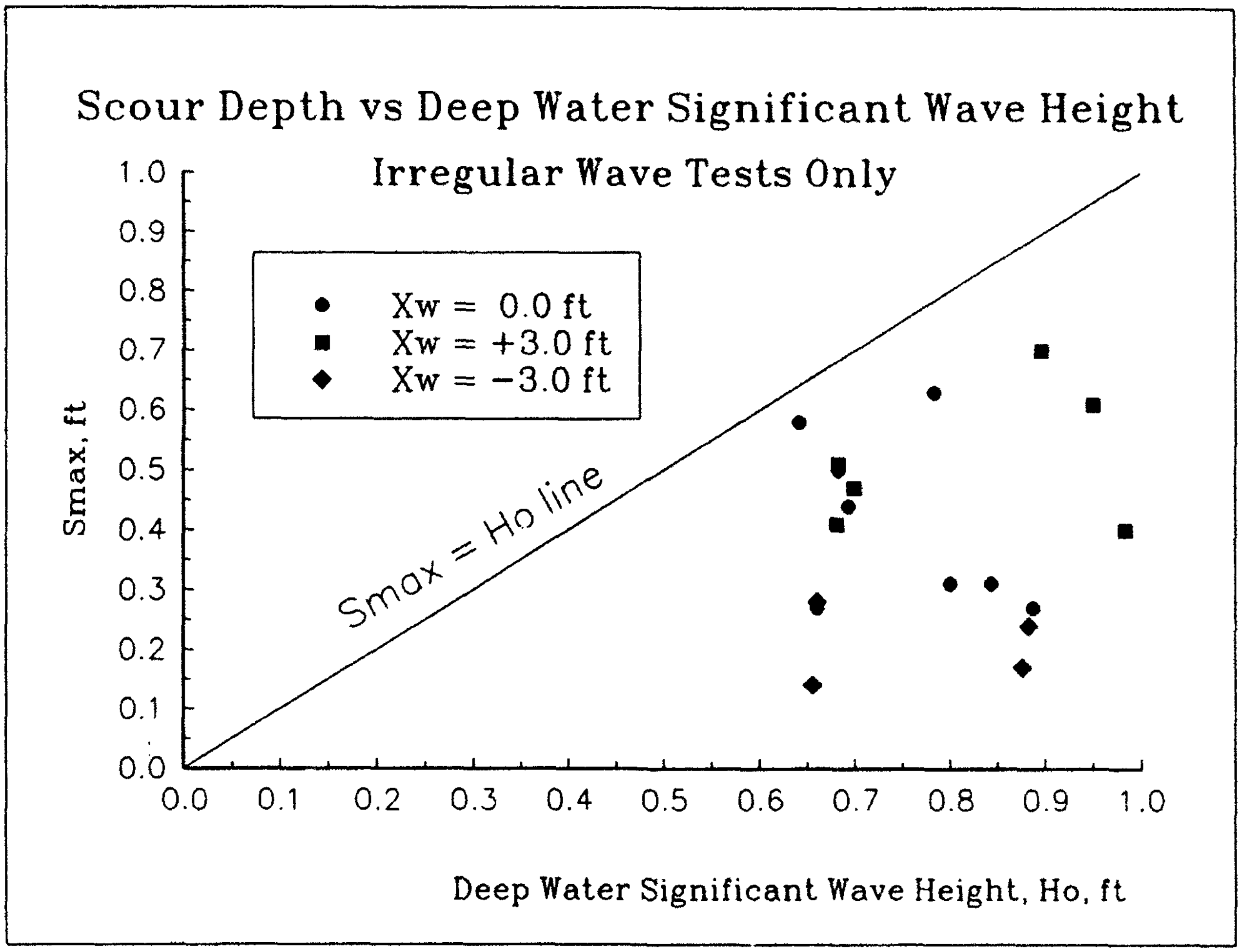


Figure 11. Plot of maximum scour depth versus deepwater significant wave height for irregular wave tests

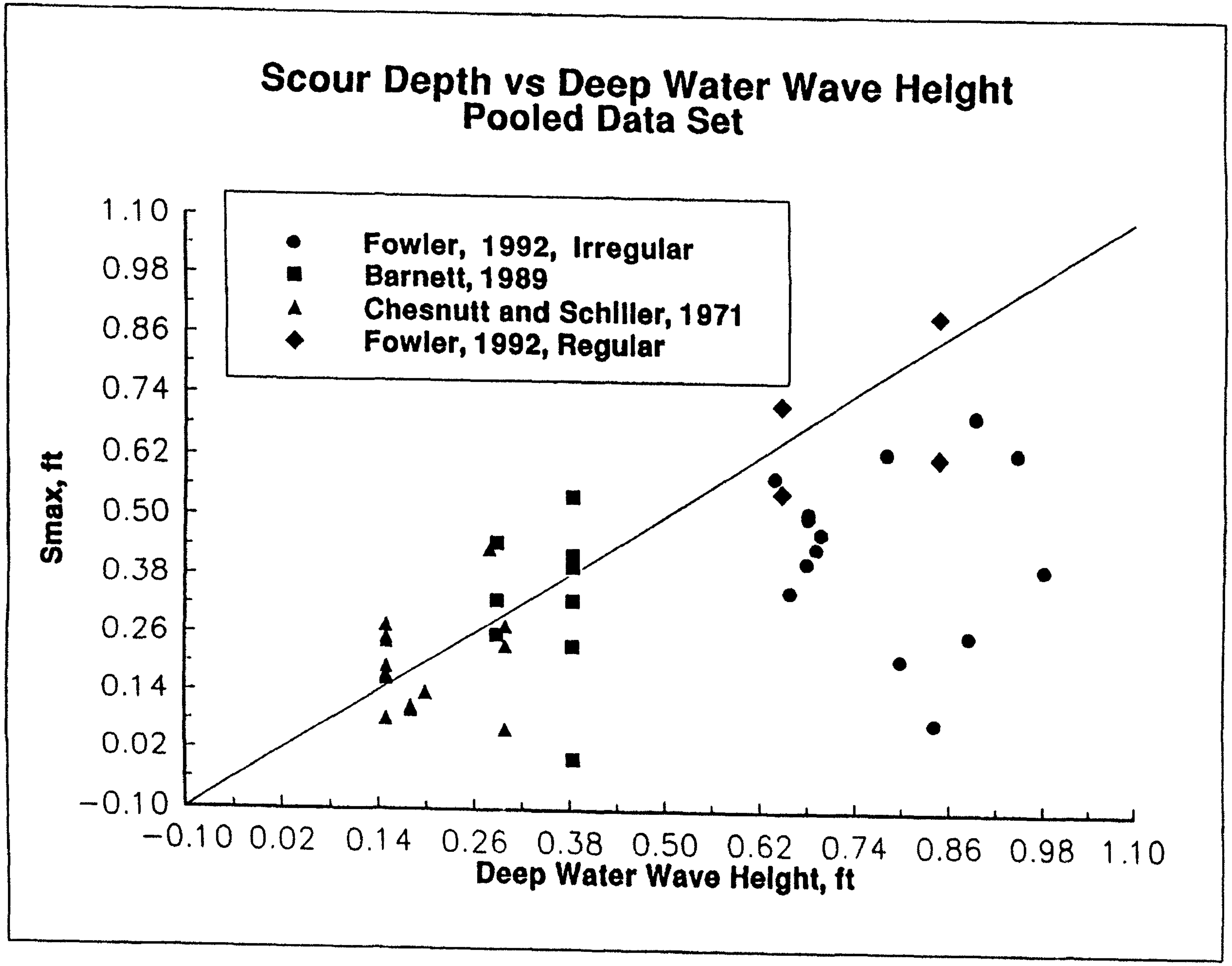


Figure 12. Combined data set of scour at vertical seawalls

Regular Versus Irregular Waves

33. The exact effect of regular versus irregular waves in this situation is not known, and to investigate this further, the present study was extended to include four cases of monochromatic waves having comparable depths, heights, periods, and seawall locations to four of the irregular wave tests. Although this is by no means a complete effort to determine the relationship between profiles generated by regular and irregular waves, some insight may be gained. In each of the regular wave cases, where H_o corresponds to the average of all waves generated, scour depths exceeded scour depths associated with the irregular wave cases, as depicted in Figure 13. On the average, the increased scour was approximately 15 percent.

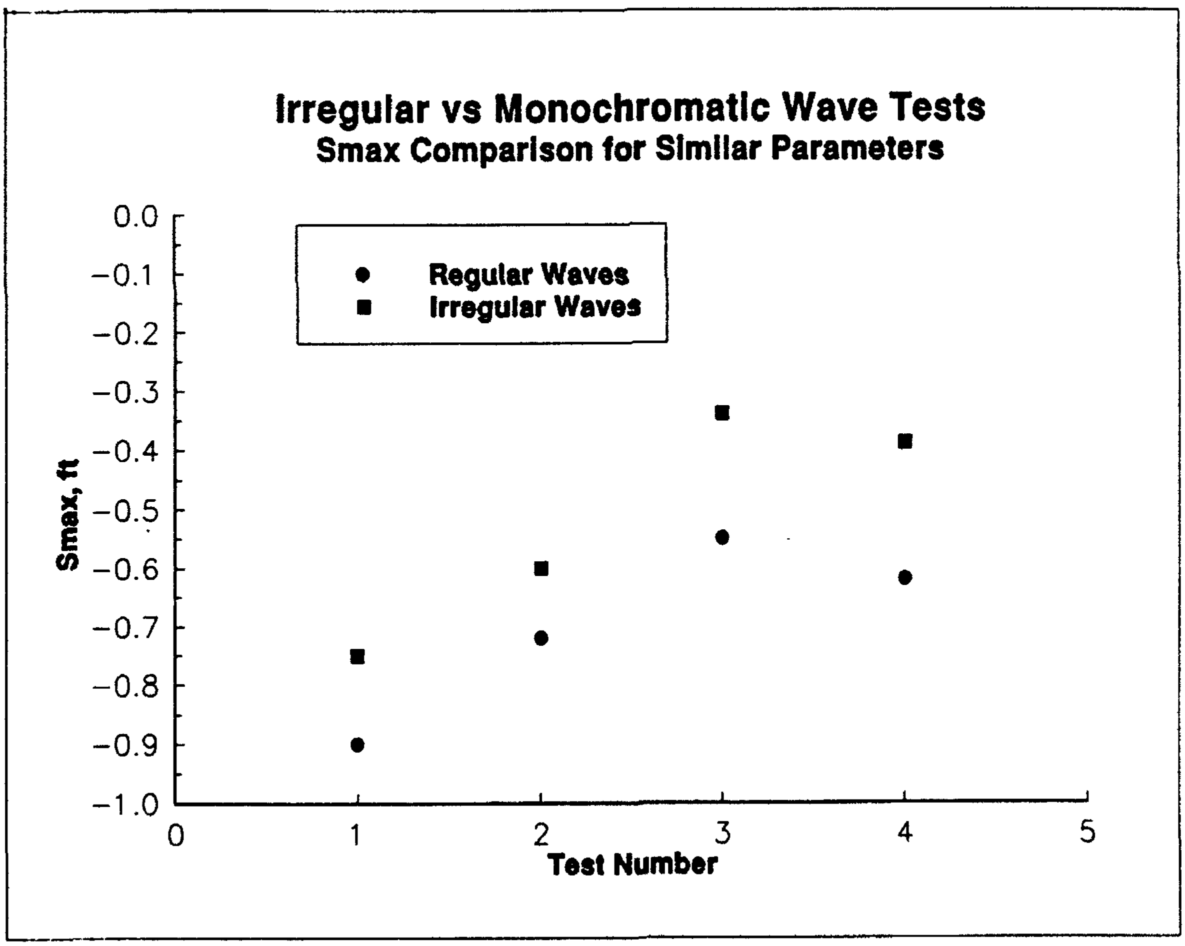


Figure 13. Plot showing difference between scour depths generated by regular and irregular waves in the laboratory

PART V: DISCUSSION AND SUMMARY

General

34. Prediction methods for scour at vertical walls may vary from using rule-of-thumb estimates to semi-empirically derived equations. The present test results were used to assess scour prediction methods previously discussed. When existing numerical methods are not adequate or sufficient, physical model studies often are performed. The following section briefly discusses the merits and shortcomings of several scour prediction techniques. Since maximum scour for seawalls impacted by nonbreaking waves is not as significant as with breaking waves, and the maximum scour depth location occurs a considerable distance seaward of the structure itself, and not immediately at the base (Hughes and Fowler 1990, Herbich et al. 1984, Xie 1981), methods for predicting this type of scour are not assessed here.

$S_{max}/H_o \leq 1$ Rule-of-Thumb Method

35. As was stated earlier, the irregular wave data support the CERC rule-of-thumb method, which states that maximum scour depth will be less than or equal to the incident unbroken wave height (see Figure 11). When combined with data from other studies, the rule-of-thumb method does not hold for several cases where monochromatic waves were used. As seen in Figure 13, there is some evidence that studies conducted using regular waves may tend to overpredict scour depths by an undetermined amount. Available data from several field studies strongly support the $S_{max}/H_o \leq 1$ rule of thumb.

Dean's Approximate Principle

36. Dean's approximate principle (eroded volume will be less than or equal to volume retained by the seawall had it not been in place) was not assessed using data from the present study. However, previous limited seawall tests by Hughes and Fowler (1990), conducted in association with efforts to validate movable-bed modeling scaling laws, tended to support the approximate principle, yielding a ratio of 1.03 eroded volume to retained volume for regular wave tests and a ratio of 0.83 for irregular wave tests. Data obtained by Barnett (1987) also support Dean's approximate principle, where 11 comparison tests yielded an average ratio of 0.61 for eroded to retained volume. The main problem with using this principle to determine scour volume is that it requires determination of beach profiles for given sediments and wave climate both prior to and subsequent to a design event. At present, this is quite difficult to accomplish, and existing prediction models such as SBEACH by Larson and Kraus (1989), though quite promising, are still relatively unproven.

Song and Schiller's Equation

37. Song and Schiller's (1971) method was used to predict maximum scour depth for the 18 different irregular wave tests conducted in this study. Powell (1987) found that "it would appear from test data that the range (of applicability for Song and Schillers equation) should be on the order of 0.5 to 1.0 = X/X_b". Results of the calculations using Song and Schiller's equation with the irregular wave data are given in Figure 14, where predicted maximum scour depth is plotted versus measured scour depth. As can be seen in the figure, predictions from this method fit the irregular wave data reasonably well. The range of values for X/X_b for the present tests is 0.67 - 1.38.

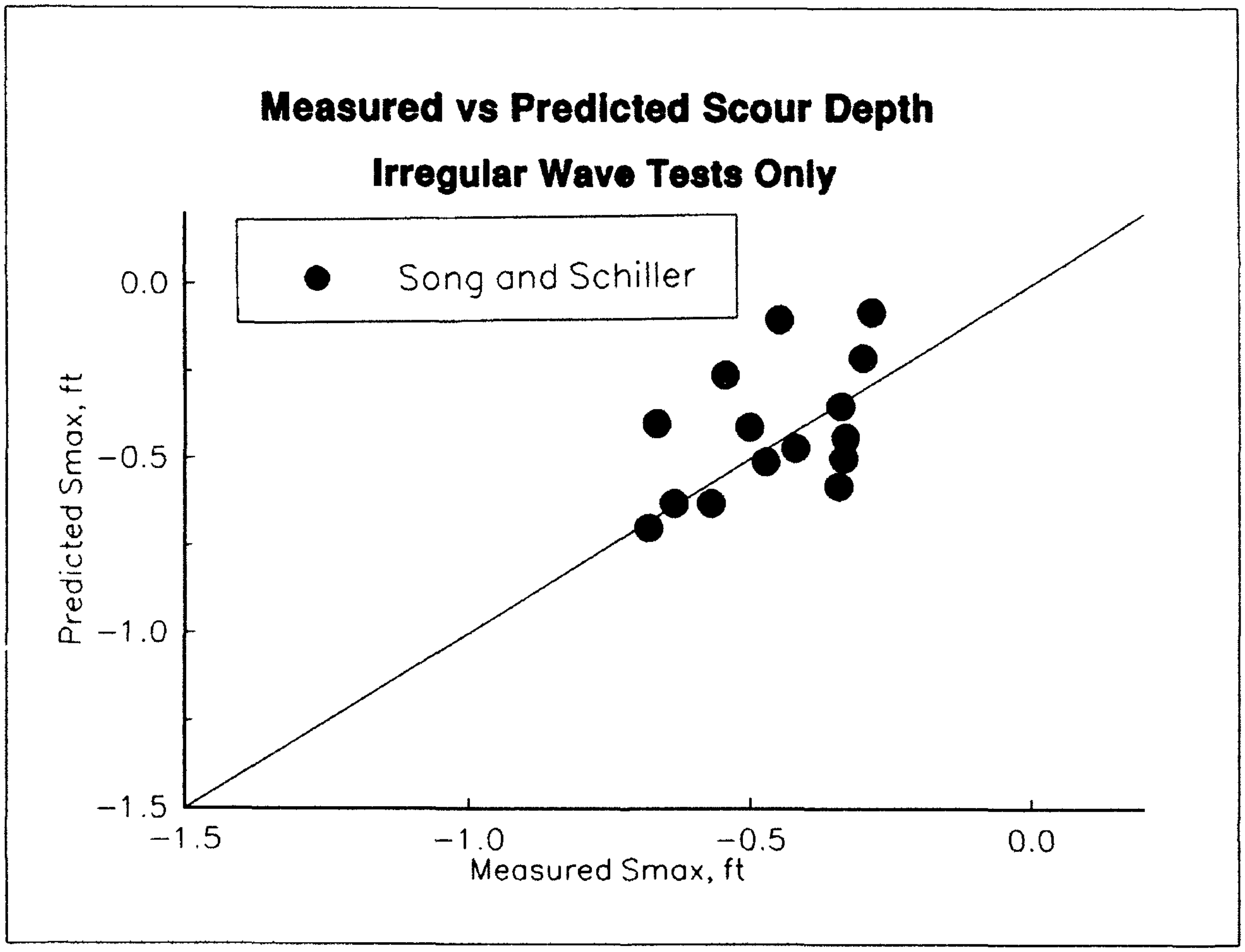


Figure 14. Predicted scour depths versus measured scour depths using Song and Schiller's equation

Jones' Equation

38. Jones (1975) used a number of limiting assumptions (including infinitely long structure and perfect reflection) to derive an equation for estimation of scour depth at the toe of vertical seawalls which relates ultimate scour depth S to breaking wave height H_b and X_s, the dimensionless location of seawall relative to mean sea level. Although the location of wave breaking was not specifically measured in the laboratory tests, values obtained from final equilibrium plots were used to closely estimate the distances required for Jones' method. These then were used in Equation 3 to compare predicted values of scour versus measured values. Results of this comparison are presented in Figure 15. One major problem with the Jones' equation is that the zero scour is predicted when the seawall is located at $X_s = 1$ (at the shoreline). This is contradicted in every study examined; in fact, some have found that this seawall location corresponds to the greatest scour condition.

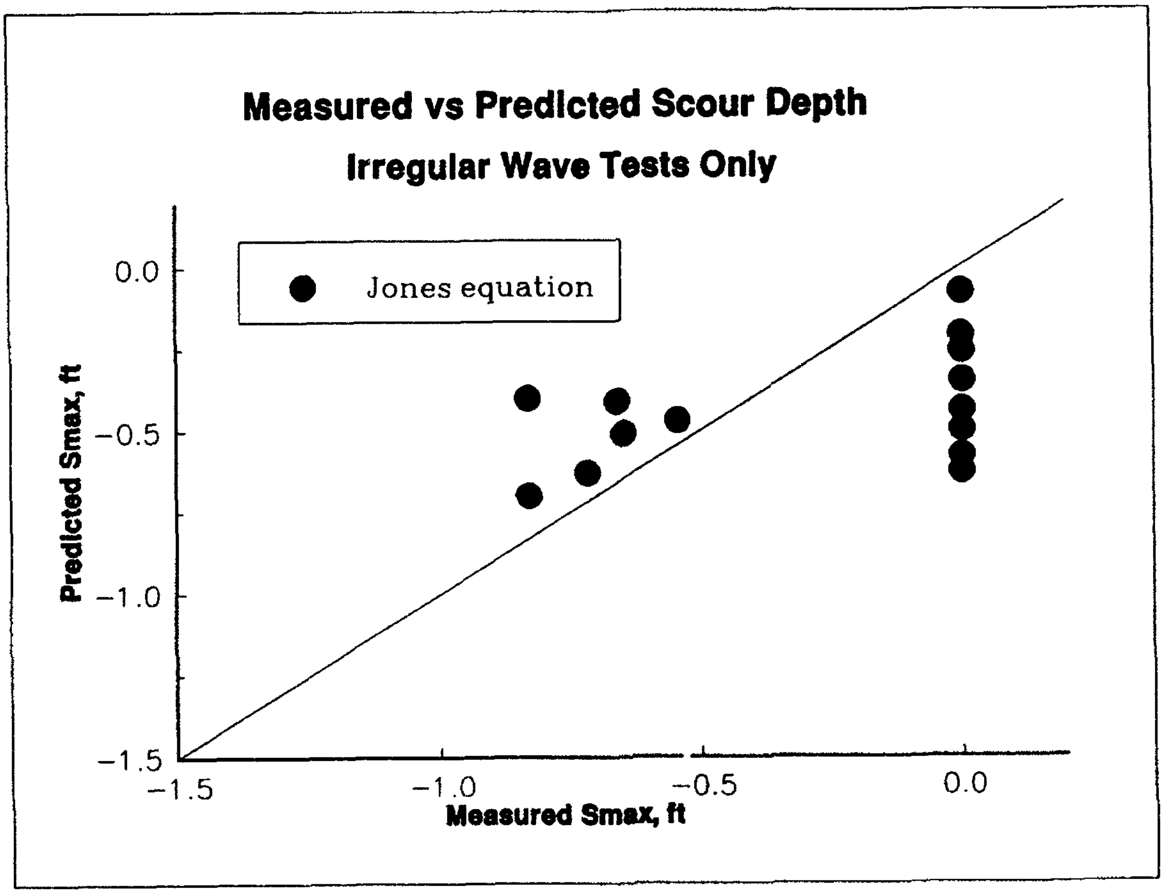


Figure 15. Predicted maximum scour depth versus measured maximum scour depth using Jones' equation

Proposed Equation

39. A statistical analysis of the irregular wave results obtained from this study indicates that ultimate scour depth is most correlated to incident deepwater significant wave height, deepwater wave length, and pre-scour water depth at the wall d_w . Since only one grain size and one initial beach slope were used in the tests, no conclusions can be drawn regarding the effects of grain size (fall speed) or initial beach slope. However, it can be argued that, for the case of a vertical wall with nearly perfect reflection characteristics, the effects of beach slope and reflections are accounted for by the presence of d_w, H_o, and L_o in the equation. Subject to the constraints shown below, the following equation for prediction of maximum depth of scour is proposed based on a mathematical analysis of the irregular wave data.

$$\frac{S_{max}}{H_o} = \sqrt{22.72\, d_w/L_o + .25} \qquad (12)$$

Use of Equation 12 is limited to cases where $-0.011 \le d_w/L_o \le 0.045$ and $0.015 \le H_o/L_o \le 0.040$. The last condition restricts the equation to use with waves that are typical of most storms. The locus of this equation is plotted, along with data obtained during this study, in Figure 16. Maximum scour depths predicted by this equation are plotted versus measured values from irregular wave tests in Figure 17.

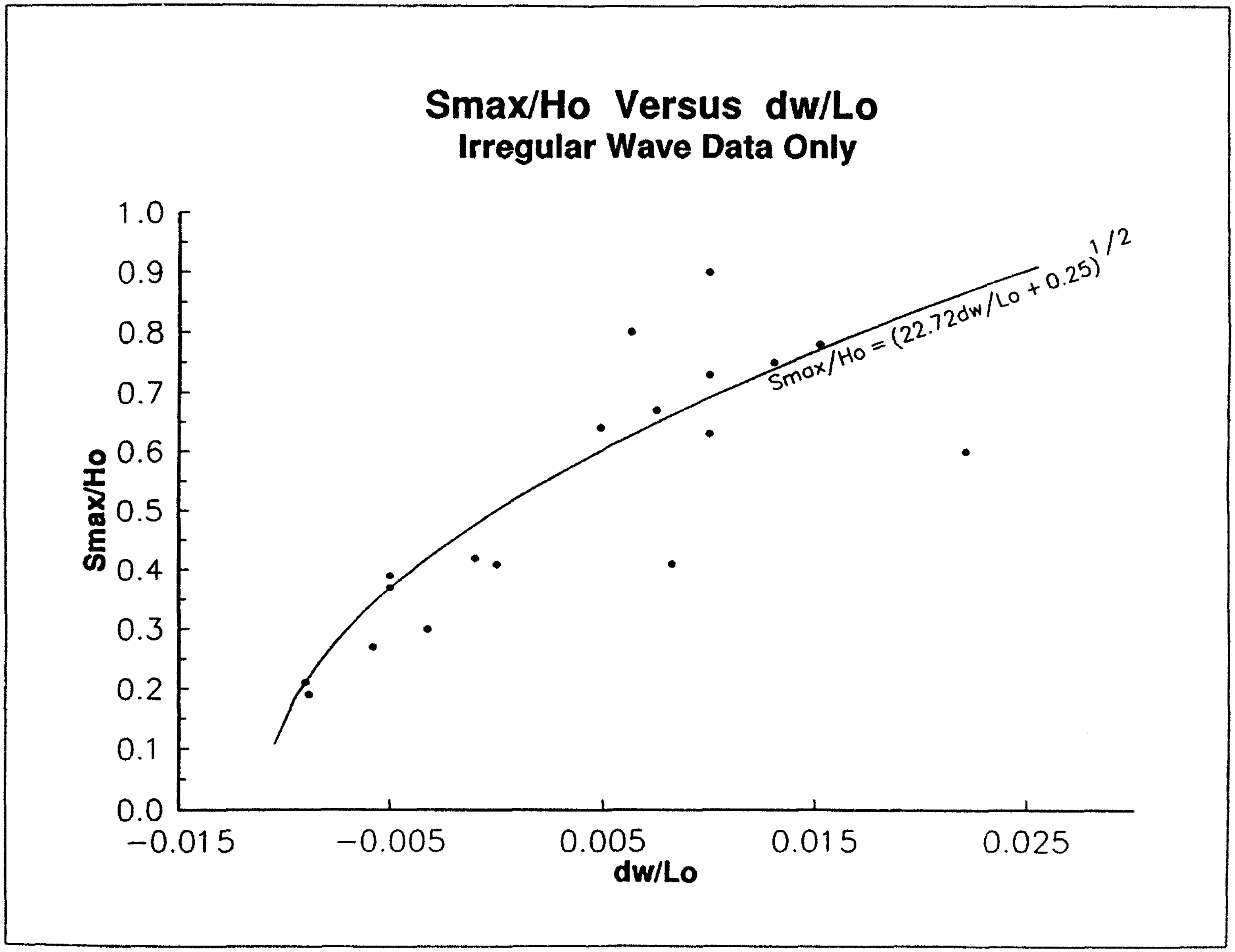


Figure 16. Relative maximum scour depth versus relative depth at seawall with Equation 12 included

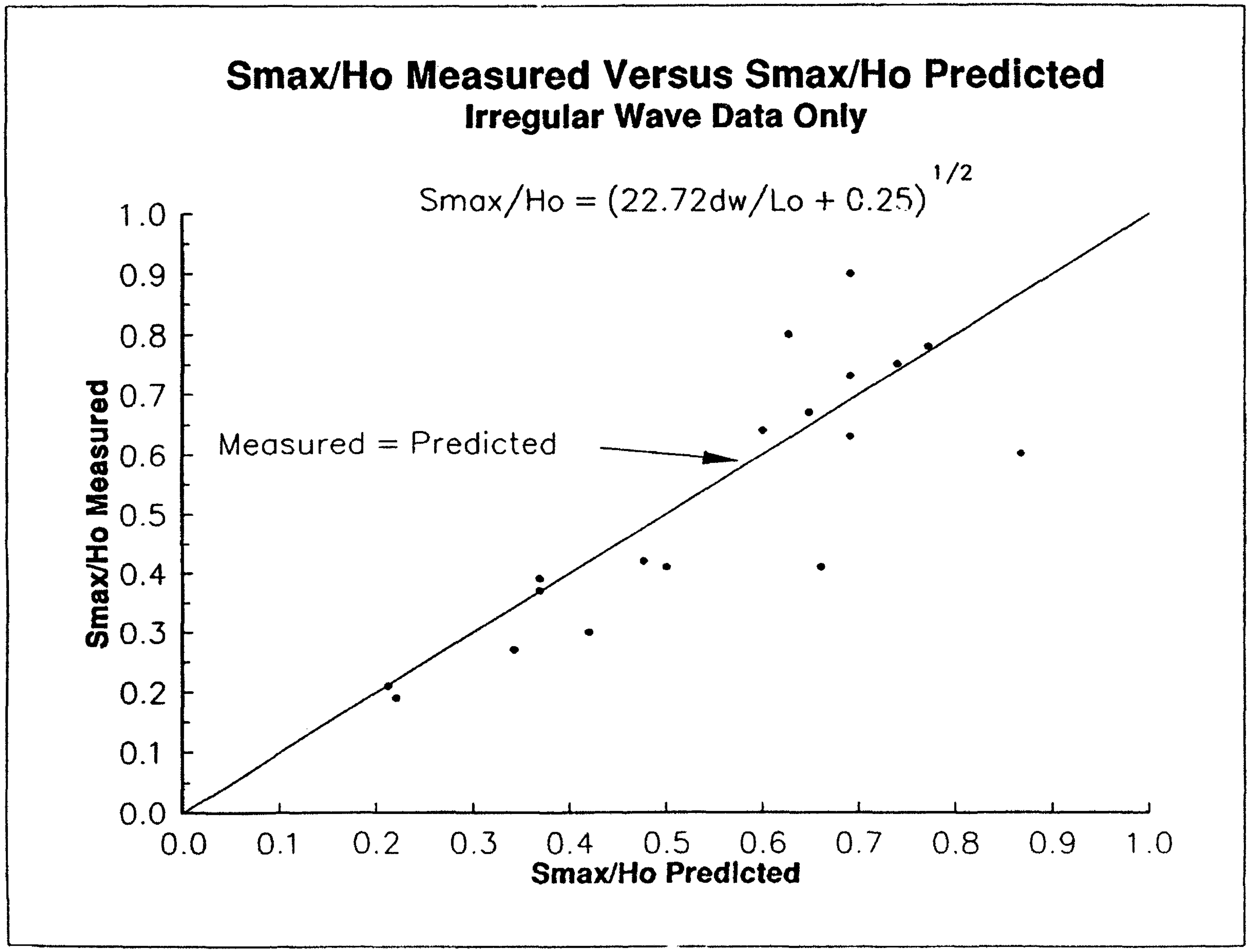


Figure 17. Predicted scour depths versus measured scour depths using the proposed equation with irregular wave data only

40. Subject to the conditions outlined for Equation 12 above, the data available for vertical (90-deg) wall tests from Barnett (1987) and Chesnutt and Schiller (1971) also were plotted for S_{max}/H_o versus d_w/L_o in Figure 18. The locus of Equation 12 also is included in this plot. With two exceptions, these data also appear to fit this curve reasonably well. The effect of these two exceptions may well be reduced when one considers the likelihood that the depths of scour were artificially large due to the use of regular waves with these studies (see discussion on regular versus irregular waves in Part II). Finally, Equation 12 is used with the pooled data to produce Figure 19, which compares predicted relative scour to measured relative scour for the combined data set used above with the added restrictions of $-0.011 \le d_w/L_o \le 0.045$ and $0.015 \le H_o/L_o \le 0.040$.

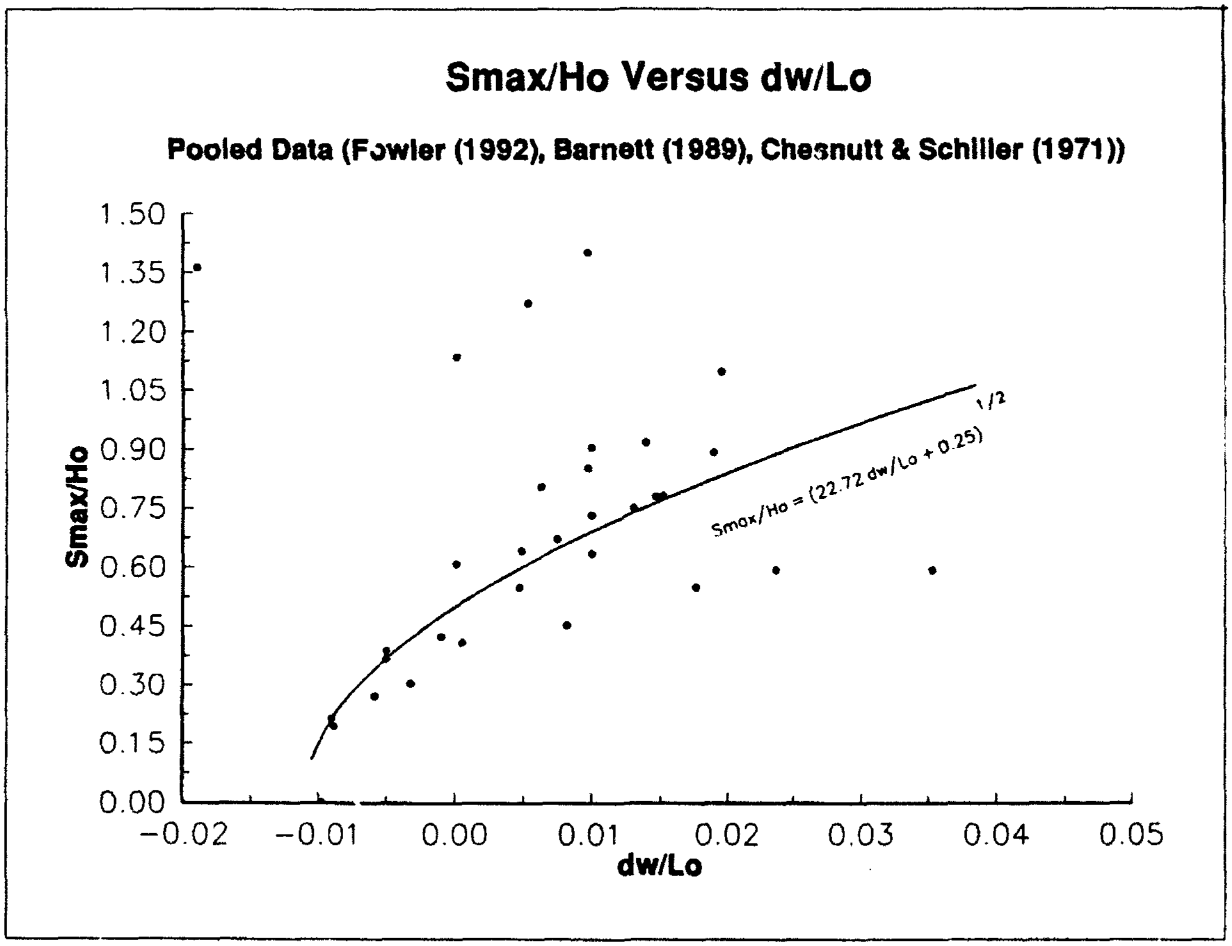


Figure 18. Relative scour depth versus relative depth at seawall with plot of Equation 12 included with pooled data set

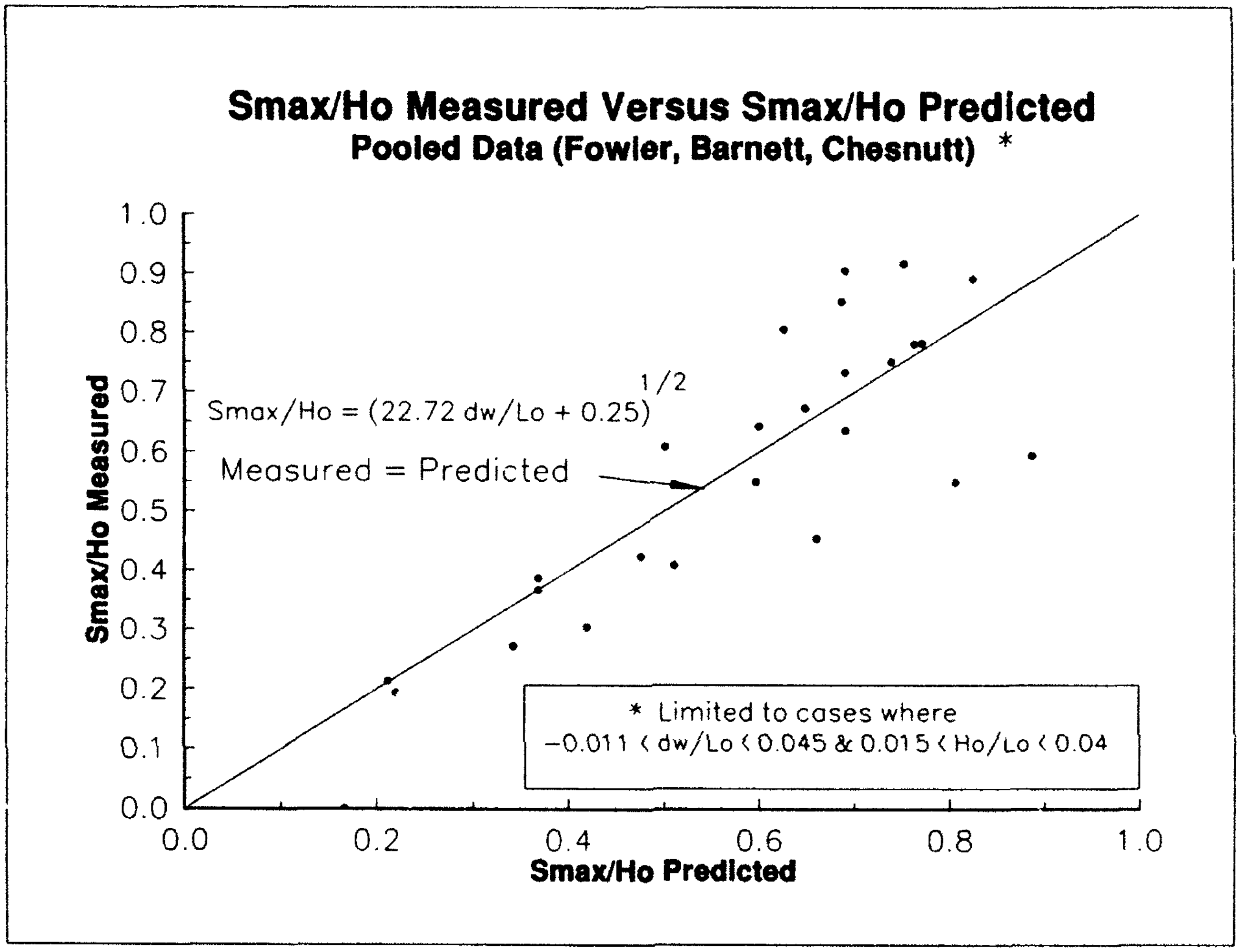


Figure 19. Predicted scour depths versus measured scour depths using Equation 12 with pooled data

Summary

41. The previous sections briefly discussed merits and shortcomings of several scour prediction techniques. In general, prediction techniques for scour at vertical walls are either rule-of-thumb methods or semi-empirical equations based on limited laboratory and field studies. Results from this study and numerous field studies tend to support the most widely used rule of thumb, which states that $S_{max}/H_o \leq 1$. Dean's approximate principle appears to be supported by numerous laboratory studies and limited field observations. A major shortcoming of this method is that it requires determination of beach profiles for given sediments and wave climate both prior to and subsequent to

a design event. At present this is quite difficult to accomplish. When used with various semi-empirical equations for prediction of S_{max}, the equation of Song and Schiller (1973) performed reasonably well within the limits of applicability given by $0.5 \leq X/X_b \leq 1$. An empirical equation based on the irregular wave data generated from this study also is proposed subject to previously described limitations.

42. For seawalls to be constructed in coastal situations where $-0.011 \leq d_w/L_o \leq 0.025$ and $0.015 \leq H_o/L_o \leq 0.04$, Equation 12 is recommended for predicting ultimate scour depth. For all other cases, the $S_{max}/H_o \leq 1$ rule of thumb should be appropriate for predicting ultimate scour depth at vertical walls.

REFERENCES

Athow, Robert F., and Pankow, Walter E. 1986 (July). "Annotated Bibliography for Navigation Training Structures," Technical Report REMR-HY-1, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Barnett, M. R. 1987. "Laboratory Study of the Effects of a Vertical Seawall on Beach Profile Response," Report 87/005, Coastal and Oceanographic Engineering Department, University of Florida, Gainesville, FL.

Chesnutt, C. B., and Schiller, R. E. 1971. "Scour of Simulated Gulf Coast Sand Beaches Due to Wave Action in Front of Sea Walls and Dune Barriers," COE Report No. 139, TAMU-SG-71-207, Texas A&M University, College Station, TX.

Dean, R. G. 1986. Coastal Armoring: Effects, Principles and Mitigation," *Proceedings of the 20th Coastal Engineering Conference*, American Society of Civil Engineers, Vol 3, pp 1843-1857.

Dette, H. H., and Uliczka, K. 1987. "Prototype Investigation on Time-Dependent Dune Recession and Beach Erosion," *Proceedings of Coastal Sediments '87*, American Society of Civil Engineers, Vol 2, pp 1430-1444.

Goda, Y. 1970. "Numerical Experiments on Wave Statistics with Spectral Simulation," Report of the Port and Harbor Research Institute, Vol 9, No. 3, Japan.

Hales, Lyndall Z., 1980. "Erosion Control of Scour During Construction; Report 2, Literature Survey of Theoretical, Experimental, and Prototype Investigation," Technical Report HL-80-3, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Herbich, J. B., and Ko, S. C. 1968. "Scour of Sand Beaches in Front of Seawalls," *Proceedings of the Eleventh Conference on Coastal Engineering*, pp 622-643.

Herbich, John B., Schiller, Robert E., Jr., Dunlap, Wayne A., and Watanabe, Ronald K. 1984. "Seafloor Scour; Design Guidelines for Ocean-Founded Structures," Marine Technology Society.

Hughes, S. A., and Borgman, L. E. 1987. "Beta-Rayleigh Distribution for Shallow Water Wave Heights," *Proceedings of Coastal Hydrodynamics '87*, American Society of Civil Engineers, pp 17-31.

Hughes, S. A., and Fowler, J. E. 1990. "Mid-Scale Physical Model Validation for Scour at Coastal Structures," *Proceedings of Coastal Sediments '87*, American Society of Civil Engineers, Vol 2, pp 1886-1900.

____________________. 1991. "Wave-Induced Scour Prediction at Vertical Walls," Technical Report CERC 90-8, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Jones, D. F. 1975. "The Effect of Vertical Seawalls on Longshore Currents," Ph.D. thesis, University of Florida.

Kraus, N. C. 1988. "The Effects of Seawalls on the Beach: An Extended Literature Review," *Journal of Coastal Research*, Special Issue No. 4, N. C. Kraus and O. H. Pilkey, eds., pp 1-28.

Kriebel, D. L., Dally, W. R., and Dean, R. G. 1986. "Beach Profile Response Following Severe Erosion Events," UFL/COEL-86/016, Coastal Oceanographic Engineering Department, University of Florida, Gainseville, FL.

Larson, M., and Kraus, N. C. 1989. "SBEACH: Numerical Model for Simulating Storm-Induced Beach Change, Report 1, Empirical Foundation and Model

Development," Technical Report CERC-89-9, USAE Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Mimura, N., Otsuka, Y., and Watanabe, A. 1986. "Laboratory Study on Two-Dimensional Beach Transformation Due to Irregular Waves," *Proceedings of the 20th Coastal Engineering Conference*, American Society of Civil Engineers, Vol. 2, pp 1393-1406.

Powell, K. A. 1987. "Toe Scour at Sea Walls Subject to Wave Action," Report SR 119, Hydraulic Research Limited, Wallingford, UK.

Sato, S., Tanaka, N., and Irie, I. 1968. "Study on Scouring at the Foot of Coastal Structures," *Proceedings, Eleventh Conference on Coastal Engineering*, London, England, Vol I, pp 579-598.

Sawaragi, T., and Kawasaki, Y. 1960. "Experimental Study on Behaviors of Scouring at the Toe of Seadikes by Waves," *Proceedings of the 4th Japanese Coastal Engineering Conference*, Japan Society of Civil Engineers, pp 1-12.

Seabergh, W. C. 1983. "Weir Jetty Performance: Hydraulic and Sedimentary Considerations, Hydraulic Model Investigation," Technical Report HL-83-5, Hydraulics Laboratory, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Sexton, W. J., and Moslow, T. S. 1981. "Effects of Hurricane David, 1979, on the Beaches of Seabrook Island, South Carolina," *Northeastern Geology*, Vol 3, Nos. 3 and 4, pp 297-305.

Shore Protection Manual. 1984. 4th ed., 2 Vols, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, US Government Printing Office, Washington, DC.

Song, Won Oh, and Schiller, Robert E. 1973 (June). "Experimental Studies of Beach Scour," C.O.E. Report No. 166, Texas A&M Sea Grant College.

Thompson, E. F., and Vincent, C. L. 1984. "Shallow Water Wave Height Parameters," *Journal of Waterway, Port, Coastal, and Ocean Engineering Division*, American Society of Civil Engineers, Vol 110, No 2, pp 293-299.

Turner, K.A., and Durham, D. L. 1980. "Documentation of Wave Height and Tidal Analysis Programs for Automated Data Acquisition and Control Systems," Miscellaneous Paper HL-80-2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Walton, T. L., and Sensabaugh, W. 1979. "Seawall Design on the Open Coast," Sea Grant Report No. 29, State University System of Florida, Sea Grant College Program.

Xie, S.-L. 1981. "Scouring Patterns in Front of Vertical Breakwaters and Their Influences on the Stability of the Foundations of the Breakwaters," Department of Civil Engineering, Delft University of Technology, The Netherlands.

APPENDIX A: NOTATION

C_D	Drag coefficient of the particle
d_{50}	Median grain diameter
D	Sediment particle diameter
d	Depth
d_w	Depth of water at vertical wall
Fr	Froude number, $V/(gL)^{1/2}$
g	Acceleration due to gravity, 9.8 m/s^2
H	Wave height
H_{avg}	Mean wave height of all waves
H_b	Breaking wave height
H_i	Incident wave height
H_o	Deepwater wave height
H_r	Reflected wave height
H_{rms}	Root mean square wave height
$H_{1/3}$	Average of the highest one third of all waves
l	Characteristic length
L	Characteristic length of structure
L_o	Deepwater wave length
N	Scale ratio
S	Scour depth (original sand elevation - scoured elevation)
S_{max}	Maximum scour depth below the natural bed
t	Time
T	Wave period
U_o	Near-bottom horizontal velocity
U_*	Local velocity parallel to the bottom
V	Velocity
X	Distance of the seawall from the point of wave breaking position of seawall relative to shoreline
X_b	Distance of the point of wave breaking from the intersection of msl with the pre-seawall beach profile
X_s	Dimensionless location of seawall relative to the mean sea level and beach profile intersection
ρ_s	Density of sediment grains
ρ	Fluid density
ω	Sediment fall speed
γ	Specific weight
γ_s	Sediment specific weight
ν	Kinematic viscosity, μ/ρ
ϕ	Bed material angle of repose

Subscripts

l	Characteristic length
max	Maximum at wall
mo	Zeroth moment
t	Time
T	Wave period
x	Distance in x direction
y	Distance in y direction
z	Distance in z direction
ω	Fall speed

Waterways Experiment Station Cataloging-in-Publication Data

Fowler, Jimmy E.
Scour problems and methods for prediction of maximum scour at vertical seawalls / by Jimmy E. Fowler, Coastal Engineering Research Center ; prepared for Department of the Army, US Army Corps of Engineers.
45 p. : ill. ; 28 cm. — (Technical report ; CERC-92-16)
Includes bibliographical references.
1. Scour (Hydraulic engineering) 2. Sea-walls. 3. Embankments. 4. Breakwaters. I. United States. Army. Corps of Engineers. II. Coastal Engineering Research Center (U.S.) III. U.S. Army Engineer Waterways Experiment Station. IV. Title. V. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; CERC-92-16.
TA7 W34 no.CERC-92-16